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POWER DIVISION

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Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

COOLING WATER FOR STEAM ELECTRIC STATIONS ON TIDEWATER[a]

By R. W. Spencer[1], M. ASCE, and John Bruce[2]

SYNOPSIS

The electric utility industry is familiar with the rapidly increasing size of steam electric stations and the enormous quantities of cooling water that must be moved to serve them. Designers recognize the importance of the many factors which must be considered in providing a system of lowest initial cost, and minimum operating expense, with a high degree of reliability and availability. This paper presents a brief review of a number of tidewater installations in California rather than an analysis of the detail of mechanical and hydraulic features of cooling water systems.

SOUTHERN CALIFORNIA EDISON COMPANY SYSTEM

As will be noted from the map (Fig. 1) all steam stations on the Southern California Edison Company system, with one exception, are dependent directly on the Pacific Ocean for cooling water. Of necessity several are situated at, or quite near to, resort areas and public beaches. State agencies and good public relations require the areas to be protected for propagation of fish, for ocean sports, for boating, and for general aesthetic enjoyment.

At four plants this has been accomplished by the installation of large concrete conduits extending well out into the open ocean and resting in excavated

Note.—Discussion open until November 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. PO 3, June, 1960.

[a] Presented at the February 1959 ASCE Convention in Los Angeles, Calif.

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trenches in the sea bottom, although the primary reason for such location has been to avoid the hazard of damage to exposed structure by heavy seas. No provision is made for unwatering or direct cleaning of the conduits but anti-fouling measures are taken. Ten years' experience with this scheme has proved it to be reliable and satisfactory although somewhat more costly than cooling water arrangements in protected bays and rivers, when such arrangements are available.

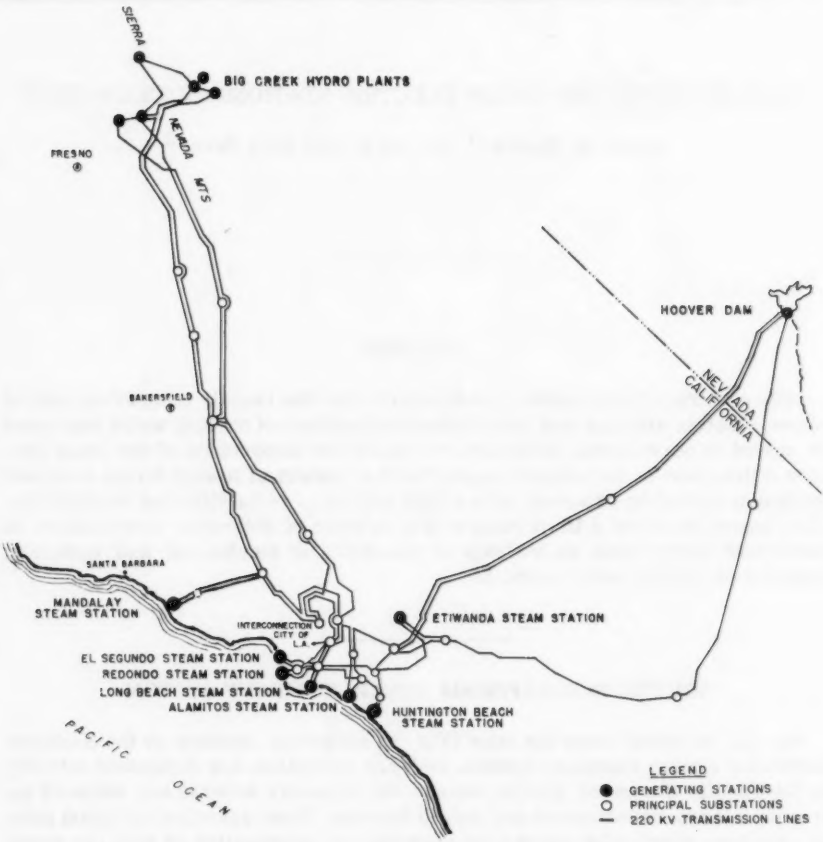
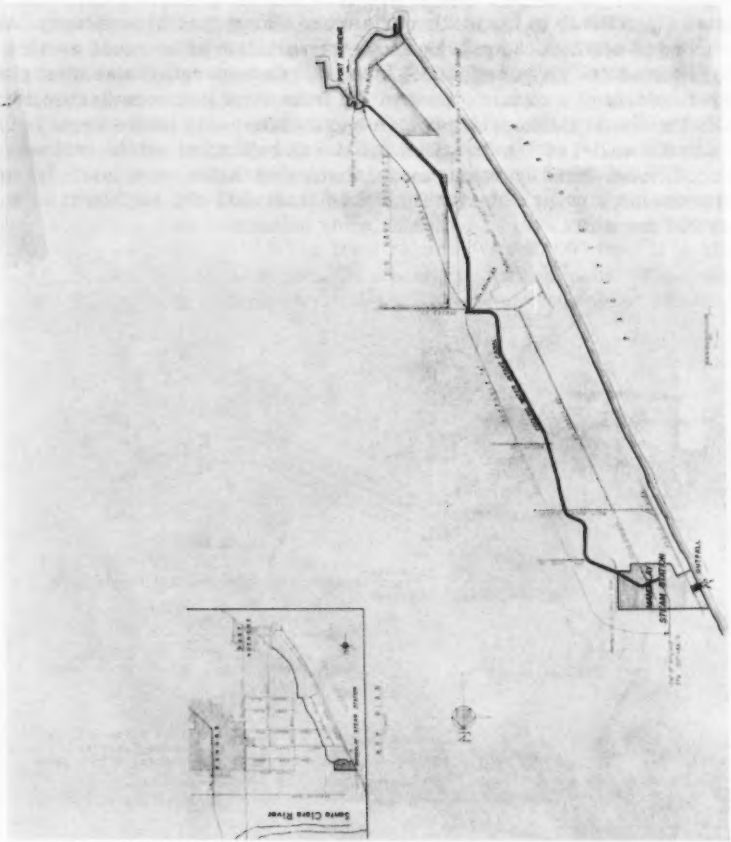


FIG. 1.—SOUTHERN CALIFORNIA EDISON COMPANY MAJOR GENERATING STATIONS

Mandalay.—The first unit at the Mandalay Steam Station is scheduled for operation in March, 1959; consequently no experience has been obtained as of the writing of this paper with the cooling water system at this station. The installation is reviewed at this time because it represents, in some ways, a new cooling water practice for this company. The station is located on Mandalay Beach about 4 miles west of the town of Oxnard and is just south of the mouth of the Santa Clara River (Fig. 2).


Irrawaddy River
Irrawaddy River
Irrawaddy River

The initial cooling water system study considered the use of conduits extending into the ocean similar to those at Redondo, El Segundo and Huntington Beach. However, the flat sea bottom gradient and the proximity of the mouth of the Santa Clara River to the station site made a long conduit necessary. Also a long record of offshore soundings showed a variation of as much as 10 ft in sand elevation at the proposed intake location. Consideration was then given to the development of a canal intake system from Port Huemene harbor, about 5 miles to the south. Because of problems associated with tidal effects in long canals, a scale model of the proposed canal was built to simulate various operating conditions. This hydraulic model confirmed that a canal could be constructed economically for a design flow of at least 800 cfs, sufficient to support four 200 mw units.

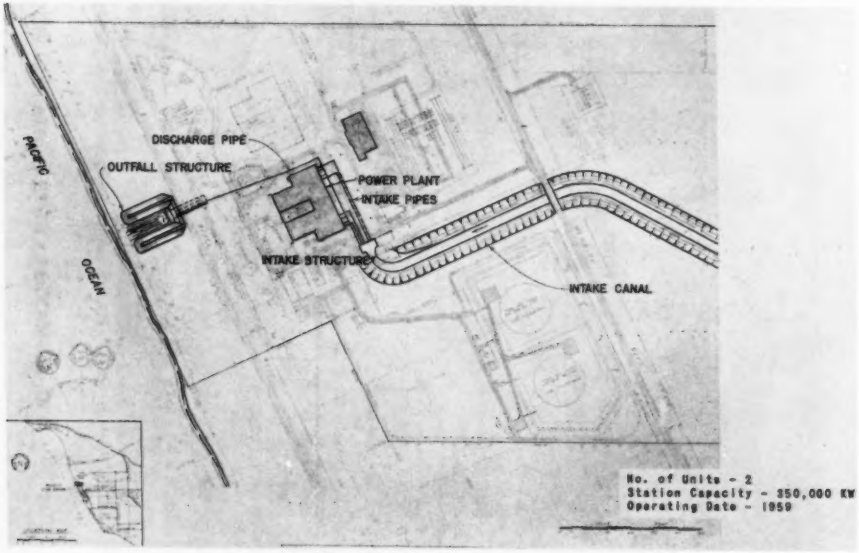


FIG. 3.—MANDALAY STEAM STATION COOLING WATER SYSTEM

The present installation will consist of two 200 mw units. Cost of the canal was about $2,000,000 less than the estimated cost for offshore submarine conduits. Approximately 1,800,000 cu yd of material were excavated in the construction. The canal was built through an agricultural area where shallow ground water level is consistently above sea level, so that there can be no seepage of ocean water into the shallow aquifer. Water is discharged through an outfall structure consisting of a rock-revetted channel extending across the beach to the mean high tide level (Fig. 3). Cost of intake canal and rock-revetted discharge channel was about $2,000,000.

El Segundo.—This station is situated in the city of El Segundo, on the ocean front. It is at the foot of sand dune bluffs and about 300 ft from the shore line. There are two units with a total capacity of 350,000 kw. The first was placed in commercial operation in 1955 and the second in 1956. The cooling water system is typical of several other of the company's installations with two 10 ft

inside-diam precast concrete conduits extending 2,600 ft and 2,100 ft along the ocean floor into the Pacific Ocean for the intake and discharge.

Terminal structures are located well beyond the area of surf disturbance in 30 ft of water at low tide. The water flows by gravity to the intake and screen structure on the site. By referring to Fig. 4, the proximity of the intake structure to the station proper can be seen. Total water flow is about 320 cfs. During the early months of operation fish swam into and concentrated in the intake fore-bay, causing some plugging of the traveling screens. Development and installation of the velocity cap has solved this problem.[3] Construction cost of the two marine conduits up to the intake structure was $2,600,000.

Long Beach.—The Long Beach Station is the oldest steam station on the company's system, now containing units installed in 1924, with additions made in 1926, 1928, 1943 and 1948. The total capacity is 400,000 kw. It is situated on Terminal Island near Long Beach on a site favorable located for obtaining a large supply of cooling water from the adjacent ship channel at relatively small expense.

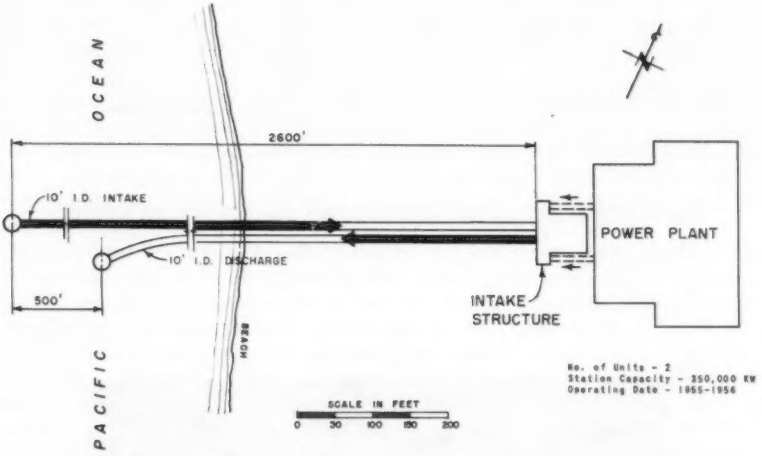


FIG. 4.—EL SEGUNDO STEAM STATION COOLING WATER SYSTEM

It is interesting to note that up to 1942 an open wooden flume carried the discharge water across the beach and into the ocean south of the property (Fig. 5). Subsequent construction of a naval base in the area required relocation of the discharge into the ship channel about 500 ft seaward of the intake (Fig. 6). Careful water temperature measurements have proven that recirculation is not significant. Cool water is drawn from a layer now about 40 ft below the surface at low tide and tidal action removes the warmed surface layer.

Redondo.—The Redondo Beach Steam Station is located on the water front near the northern limits of the city of Redondo Beach. The station consists of two separate plants, with separate but interconnected cooling water facilities common to both. There are six generating units with a total capacity of 638,000 kw. (Fig. 7). The first unit was placed in service in 1948 and the sixth unit in

[3] "Ocean Cooling Water System for 800 MW Steam Power Station," by R. H. Weight, Proceedings, ASCE, Vol. 84, No. PO 6, December, 1958.

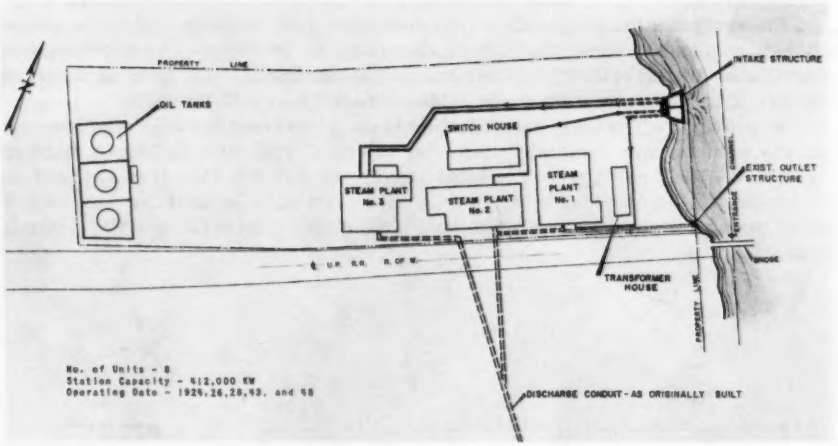


FIG. 5.—LONG BEACH STEAM STATION COOLING WATER SYSTEM

FIG. 6.—LONG BEACH STEAM STATION

1957. Circulating water enters through underwater vertical entrance type chambers located approximately 1,700 ft offshore south of the yacht basin breakwater, and flows to the station by gravity through two 10 ft diam reinforced concrete pipe conduits buried beneath the ocean floor.

Velocity caps over the intake conduit entrance chamber aid in keeping fish out of the system (Fig. 8). Water is returned to the ocean through two 10 ft diam conduits which terminate about 2,000 ft from the shore line on the north side of the breakwater. The breakwater provides a positive barrier against recirculation, and also serves to protect the intake from sand accretion. A cross tunnel supplies circulating water to the plant 1 screen well. Warmed water from both plants is normally passed through the discharge chamber at the screen well for plant 1. However the conduits are arranged so that the circulating water system for both plants may be operated as completely independent systems.

Total flow through the system is about 720 cfs. Periodic reversal of flow is practiced to control marine growth through the effect of elevated temperatures. Total costs for installation of the four marine conduits alone amounts to $4,400,000. The system including fouling control has been so satisfactory that the only maintenance expense has been the cost of an annual inspection by a diver.

Huntington Beach.—Located on the coast 30 miles south of Los Angeles, this station is the latest addition to the company's system (Fig. 9). The cooling water system is designed to support four 200 mw units, two of which are in service. Operation of the first unit started in June, 1958. Maximum design water flow for four units is 792 cfs. Except for size the cooling water system follows the same general design used at Redondo and El Segundo.

14-ft internal diam intake and discharge conduits extend about one-half mile into the ocean with about 500 ft separation between the terminal points to minimize recirculation. Construction of the marine conduits cost about $4,100,000.

Alamitos.—The Alamitos Steam Station of the Southern California Edison Company is of interest because the excellent inland site location permitted an inexpensive cooling water arrangement without the use of long marine conduits extending into the ocean. Although situated several miles inland, it is a tidewater station. The site utilizes Alamitos bay and the Los Cerritos Drain, a storm channel near the west property line, as a source. The discharge is to the San Gabriel River which parallels the east property line (Fig. 10).

To accommodate the large flow to the plant the Los Cerritos Drain was widened and deepened by the Los Angeles County Flood Control District at Edison's expense. The short intake canal is excavated in the natural soils and is an unlined channel except for the turnoff from the Cerritos drain which is riprapped (Fig. 11). It has a bottom width of 56 ft, side slopes of 2-to-1 and a bottom elevation 11 ft below mean low low water. Water at lowest tide is estimated to be 5 ft deep, and 16 ft deep at highest flood. A floating plastic boom across the canal entrance serves to prevent entrance of surface debris and also supports the property fence.

A single 96 in. precast concrete discharge line penetrates the dike along the San Gabriel River and connects to the outfall structure which is flush with the river bed. The system is designed to support a 1,000,000 kw development. Two 175,000 kw units were placed in operation during 1956 and 1957. Only one problem has developed to date (1959). This problem relates to the quantities of debris, "moss," and tumbleweeds that collect in front of the floating boom.

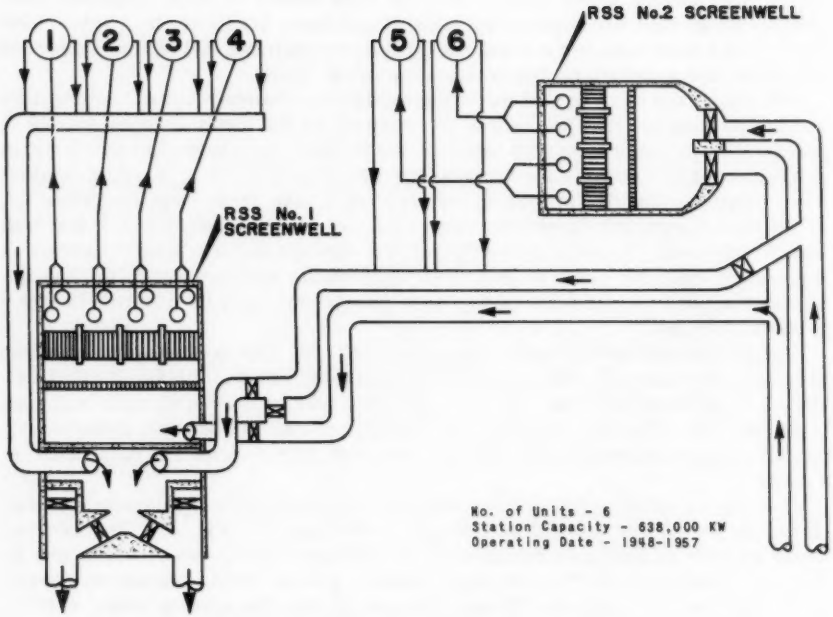


FIG. 7.—REDONDO STEAM STATION COOLING WATER SYSTEM

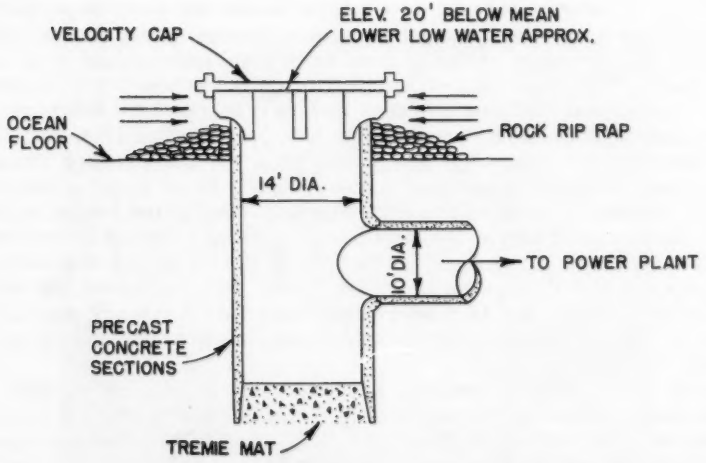


FIG. 8.—REDONDO STEAM STATION SUBMARINE CONDUIT
INLET STRUCTURE WITH VELOCITY CAP

Permanent facilities have been built to aid in this trash removal. Cost of canal, boom, and discharge headworks was about $250,000.

PACIFIC GAS AND ELECTRIC COMPANY SYSTEM

Introduction.—The Pacific Gas and Electric Company system is of special interest because it ranks as one of the largest in this country. In reviewing the various installations it becomes apparent that the area is favored with a number of excellent sites (Fig. 12) away from the exposed coast which have been fully exploited by the designers to obtain economical cooling water without the necessity of resorting to the long, expensive marine conduits that are found further south.

The preservation of fish life has been considered, in the design of a number of installations, a very important factor in public relations. The fish mortality rate has been greatly reduced by designing for uniform velocities through

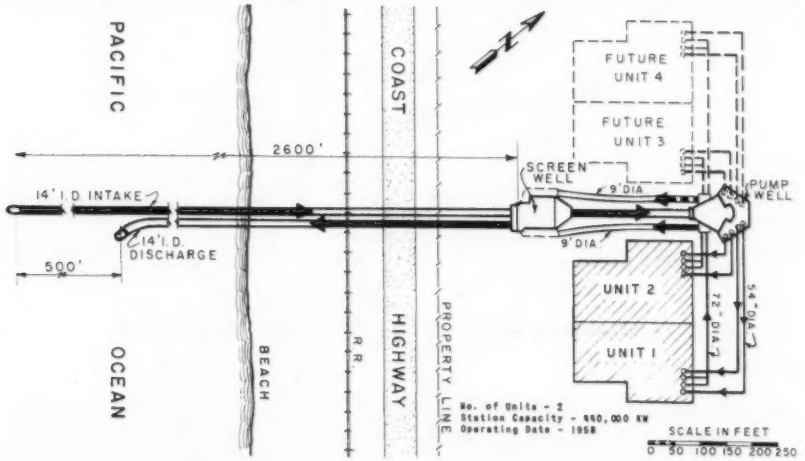


FIG. 9.—HUNTINGTON BEACH STEAM STATION COOLING WATER SYSTEM

the system, for minimum turbulence at transitions, and by limiting the duration of the exposure of fish to the warmer discharge water.

Pittsburg.—Pittsburg is presently the largest steam station in the west with a capacity of 660,000 kw in four units. Two additional 325,000 kw units that are presently (1959) under construction will increase the total station capacity to 1,310,000 kw. The plant is situated on the southern shore of the upper part of Suisun Bay, into which the Sacramento and San Joaquin rivers flow, providing a large supply of cooling water (Fig. 13).

Circulating water intake and discharge structures are located directly on the shores of the bay and are separated by about 900 ft. The discharge outfall has been located at the upstream end of the property where it was determined that the warm discharge water would be carried well out into the main stream and bypass the intake. A tidal range of 8.5 ft occurs at the plant site, and water temperatures range from 47° to 73°.

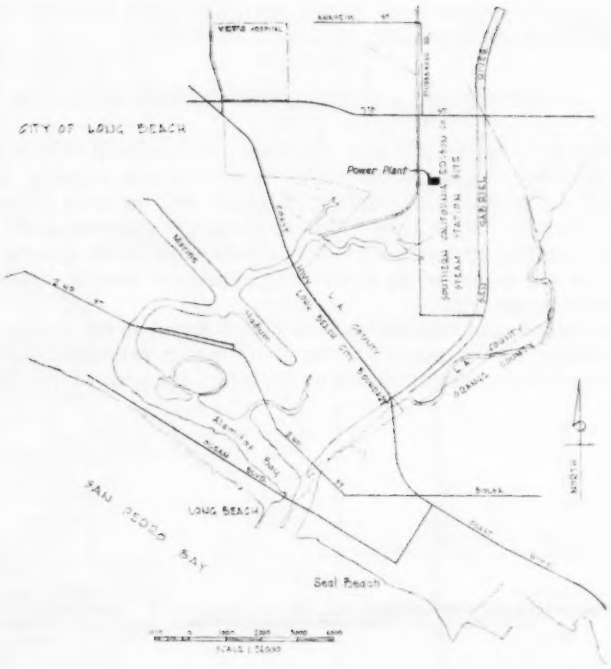


FIG. 10.—ALAMITOS STEAM STATION VICINITY MAP

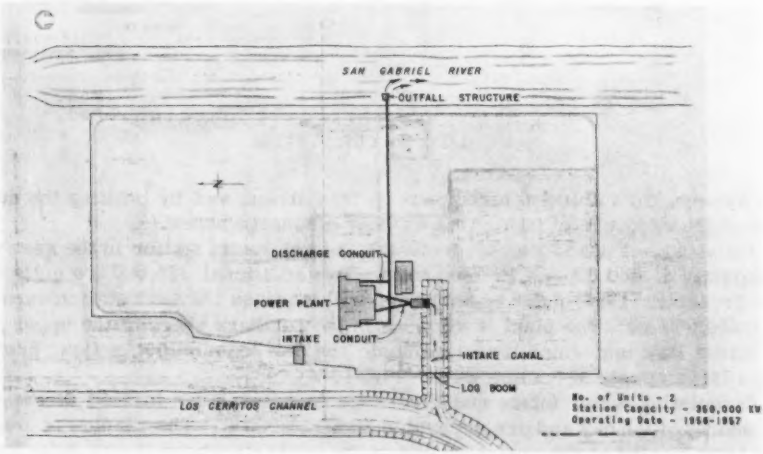


FIG. 11.—ALAMITOS STEAM STATION COOLING WATER SYSTEM

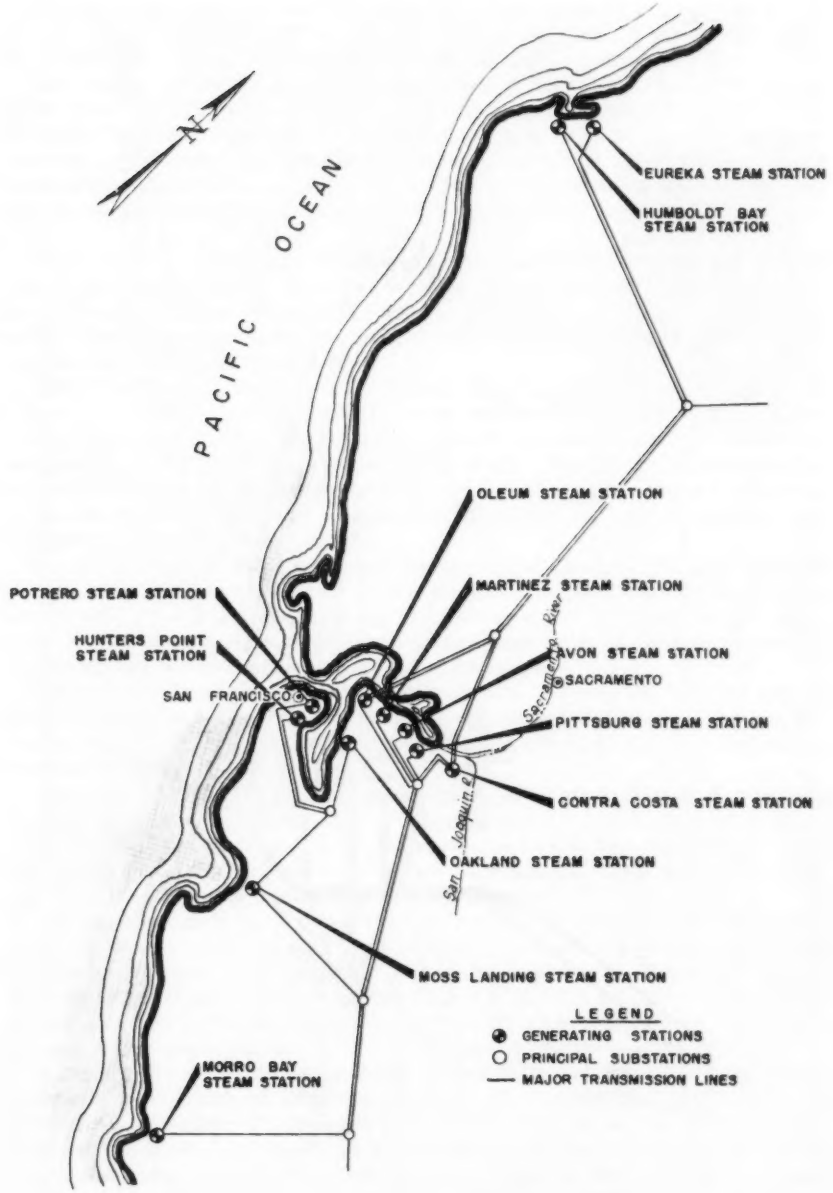


FIG. 12.—PACIFIC GAS AND ELECTRIC COMPANY
MAJOR STEAM GENERATING STATIONS

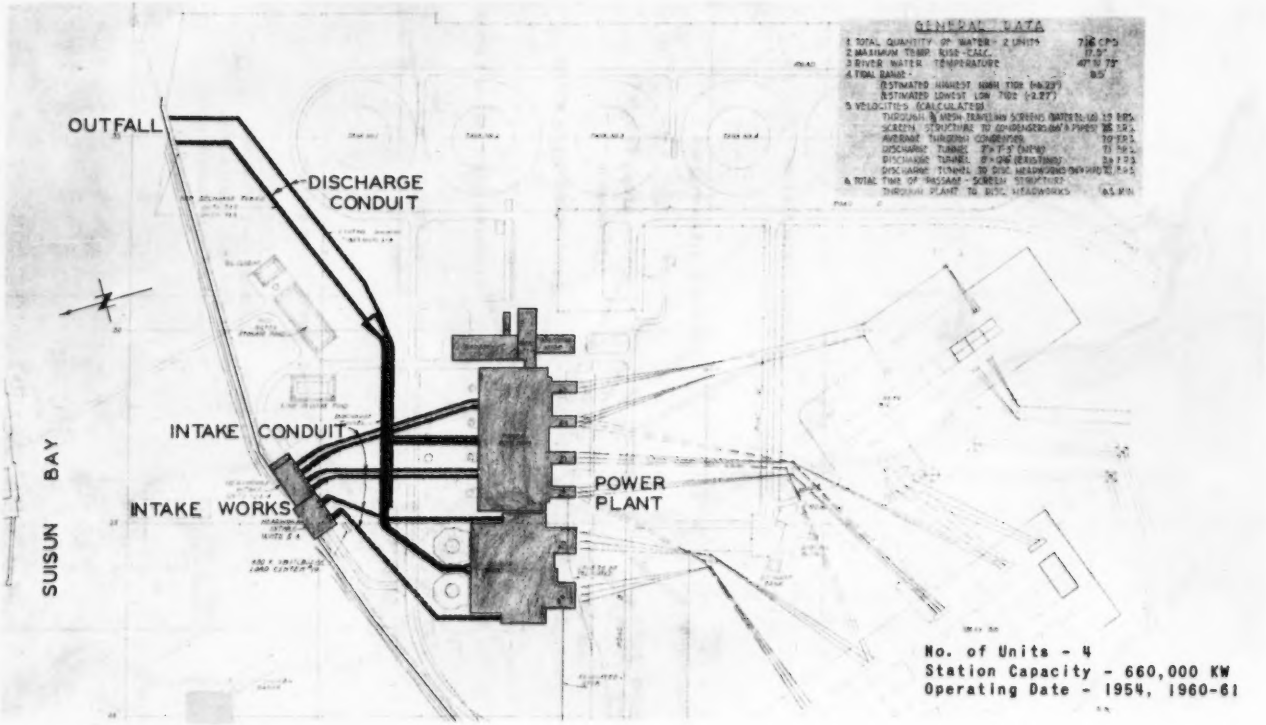


FIG. 13.—PITTSBURGH STEAM STATION COOLING WATER SYSTEM

Cooling water requirements for a station of this capacity are very large. A total of 1,622 cfs of water will be pumped through the system when the six units are in service. This is equivalent to about one billion gallons per day.

The bottom of the intake structure is set at El. -15 ft 5 in. or 14 ft 3 in. below mean low low water. Entering water velocity is between 1.5 and 2.0 fps. Silting of the intake channel and floating islands of peat requires occasional dredging of the area. The troublesome fish problem that developed at Contra Costa has been overcome at Pittsburg by placing the trash rack and traveling screens at the intake structure on the shore of the bay instead of close to the plant.

Moss Landing.—The Moss Landing Plant consists of five units with a total capacity of 575,000 kw and is located at Moss Landing Harbor in Monterey Bay. The first three units were placed in service in 1950. Two additional units were added in 1952. The site selection on protected water has permitted a very compact cooling water installation (Fig. 14).

Water enters the intake on the east and flows by gravity through two 9 ft 4 in. by 10 ft 7 in. tunnels about 350 ft to the screen and pump structure. Flow at the entrance of the intake structure is limited to the lower 11 ft. This is, presumably, to take advantage of the colder water. The two intake tunnels are not included in the recirculation system provided for mussel control; however, additional cross section was provided in the design to provide margins for anticipated normal mussel growth.

Manual cleaning has not been necessary to date. On occasion large schools of anchovies have been drawn into the system and became concentrated in the screen well structure, causing plugging of screens and affecting station capacity. Note that in the design of later stations this problem was overcome by combining the screen well with the intake structure.

Discharge is to the north into Elkhorn Slough. A lip at the 'leaving end' of the outfall structure tends to direct the flow toward the surface. The low entrance velocities at the intake have not caused any noticeable sand movement and to date dredging has not been necessary.

Morro Bay.—The Morro Bay Station is situated on the Pacific Ocean, half way between San Francisco and Los Angeles and 13 miles northwest of San Luis Obispo. The plant site is at the north end of the bay near Morro Rock. The initial installation of two 156,250 kw units was placed in service in 1955, and the site can accommodate a total of eight units.

The cooling water system is of interest because of the long discharge tunnel (Fig. 15). Water is taken from Morro Bay through an intake structure located at the edge of the bay and pumped 700 ft to the condensers. From the condensers the water is conveyed through a single 7 ft by 10 ft concrete culvert 2,850 ft long and discharged on the northerly side of Morro Rock into the Pacific Ocean. An effective barrier is thereby provided against recirculation.

Although the selection of the long discharge tunnel was based largely on economic considerations, other factors also affected the final selection.[4] One alternative considered discharging directly into the surf to the north of Morro Rock. Clam beds along the coast precluded this action. A second alternative was to return the water directly into Morro Bay. An evaluation of the estimated increase in cooling water temperatures and the resulting decrease in turbine capacity and efficiency proved the present installation to be more economical.

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4 "Civil Engineering Features of the Morro Bay Station," by J. G. Thon and Gordon L. Coltrin, Transactions, ASCE, Vol. 123, 1958, pp. 207-232.

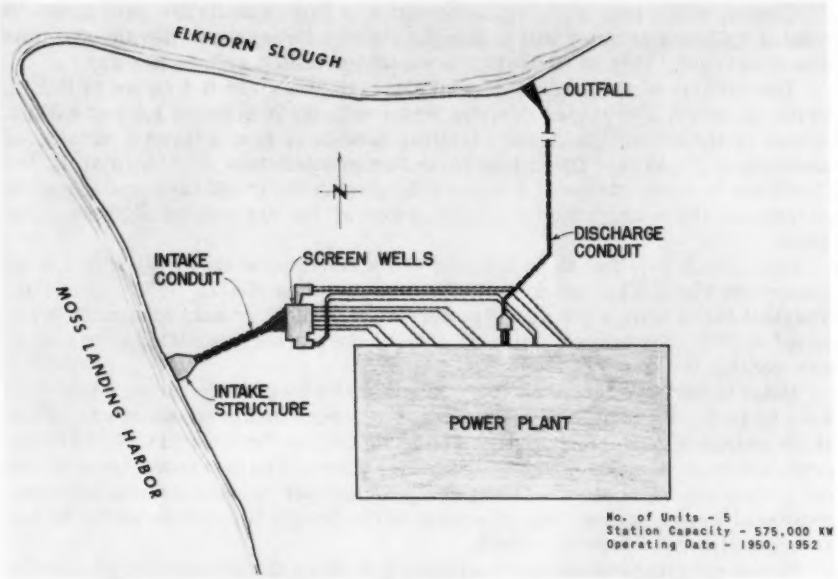

FIG. 14.—MOSS LANDING STEAM STATION COOLING WATER SYSTEM

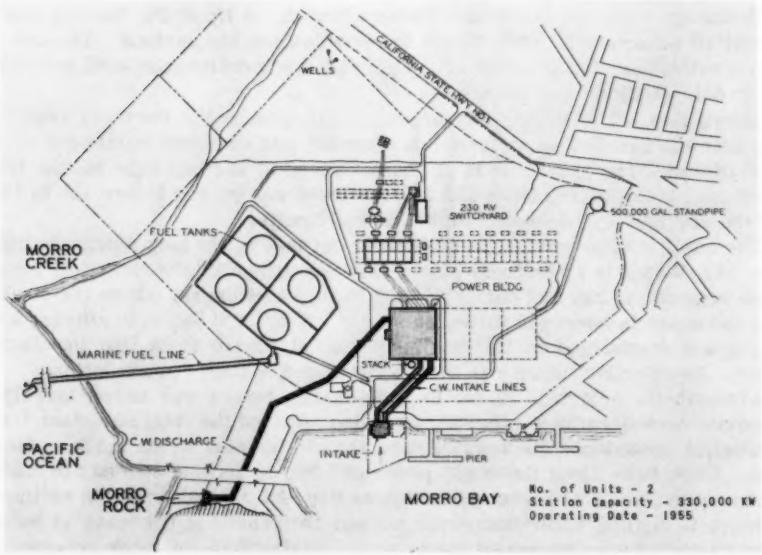

FIG. 15.—MORRO BAY STEAM STATION COOLING WATER SYSTEM

The possible effect of higher water temperature in the bay on an oyster industry in the vicinity was also a factor. The elevation of the long discharge tunnel is such that it remains full under all tidal conditions. A surge chamber is built into the system to protect it from over-pressure when accelerating the large mass of water in the tunnel during a unit start-up. Provision is made for periodic recirculation between the condenser discharge and the intake for mussel control. Occasional dredging is necessary of the area at the mouth of the intake to remove sand accretions.

Hunters Point.—Located on San Francisco Bay, this station consists of four units with a total capacity of 420,000 kw. The installation affords an opportunity for comparing design concepts in a single station covering a period of 29-yr. The first unit was installed in 1929; two more units were added in 1948, and a fourth in 1958. Intake facilities are common to the first three units (Fig. 16), while separate intake and discharge structures were added for unit 4. A channel dredged to -14 ft serves both intakes. Water velocities are below one fps and silting of the channel has not so far materialized.

The arrangement of the cooling water system for the early units is typical for that period, with gravity flow to pumps at the condensers. Fouling of intake tunnels by marine organisms was expected and manual cleaning is routine. The unit installed in 1958 has screens and pumps at the intake structure with facilities for warm water recirculation to control marine growth. A short tunnel connects to the outfall structure on the shore line. The excellent location of this site has been used to advantage in designing a relatively simple and economical cooling water installation.

Contra Costa.—The first three units of this large, modern station were placed in operation in 1951. Two additional units were added in 1953, giving a total capacity of 575,000 kw. The site is situated near the mouth of the San Joaquin River, 2-1/2 miles east of Antioch. Tidal conditions prevail, with an extreme range of 8 ft at the plant site. Cooling water requirement is 863 cfs and river temperatures range from 45° to 74°. Water is withdrawn from the river (Fig. 17) near the bottom through an intake structure 410 ft offshore and flows through two 12 ft diam conduits to the screen structure.

The entire conduit structure is supported on piles under the intake and at conduit junctions. A major operation during construction was the fabrication at the site and placement of the conduit sections, each 100 ft long and weighing 480 tons. Water is returned to the river through a discharge channel about 250 ft long; a weir across the channel and about 200 ft downstream from the discharge headworks is effective against erosion.

The intake is in the area formed by the confluence of the San Joaquin and Sacramento rivers, and the delta consists of many cross channels and tule marshes. Large masses of floating peat occasionally collect in the channel and become a hazard. These require removal by dredge. Soon after plant operation began it became apparent that fish, principally young striped bass, entering the intake conduits became trapped in the screen structure and accumulated in large quantities. Mortality was naturally high. After considerable research and investigation a trap and pumping system was devised which returns the fish to the river unharmed.[5]

Humboldt Bay.—The Humboldt Bay Station is the most recent steam plant addition to this Company's system. It is situated at Buhne Point in Humboldt Bay, seven miles south of Eureka, and consists of two 50,000 kw units. The first of the two units was placed in service in 1956 and the second in late 1958.

[5] Fish Bulletin No. 92, California Div. of Fish and Game.

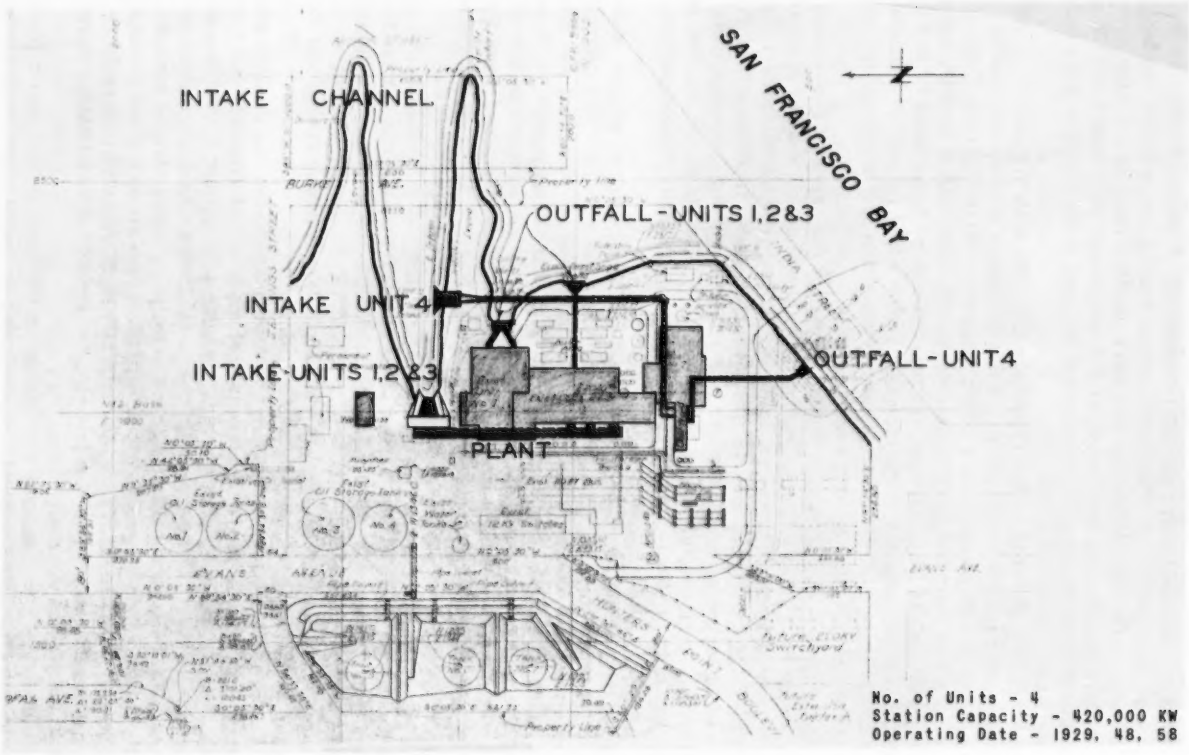

FIG. 16.—HUNTERS POINT STEAM STATION COOLING WATER SYSTEM

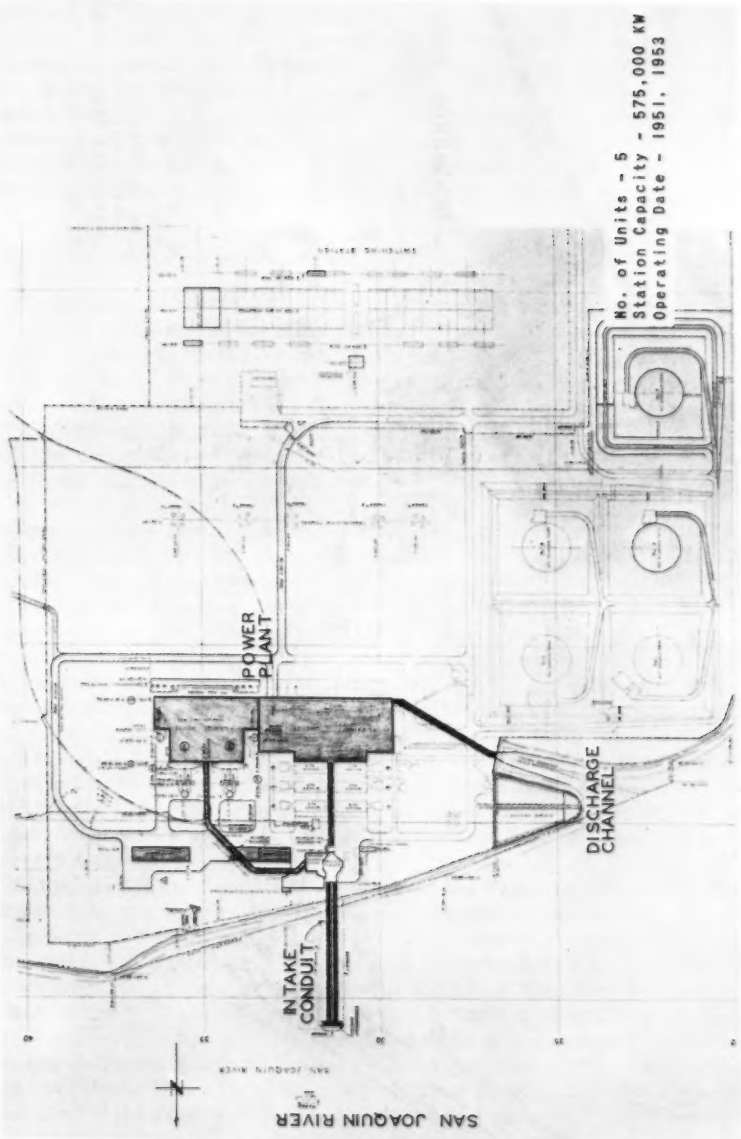

FIG. 17.—CONTRA COSTA STEAM STATION COOLING WATER SYSTEM

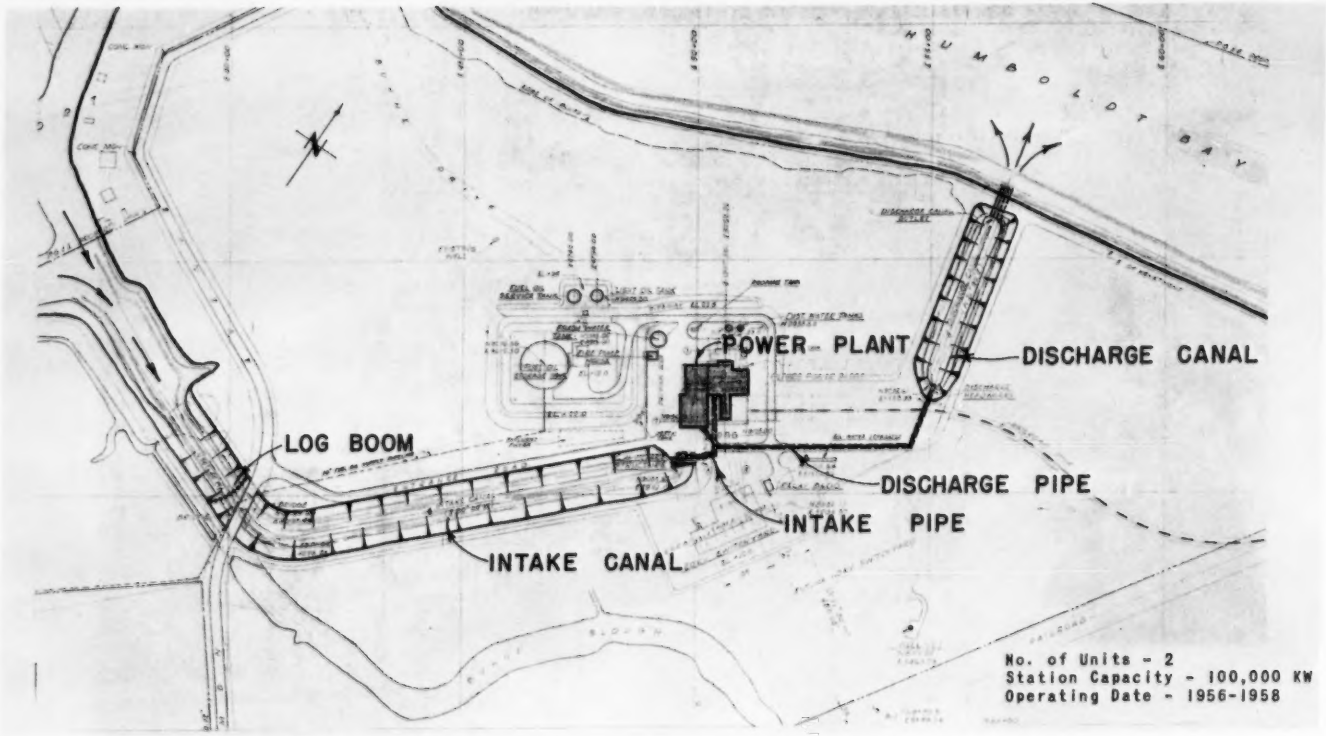

FIG. 18.—HUMBOLDT BAY STEAM STATION COOLING WATER SYSTEM

Water flows from Humboldt Bay on the west through a natural channel which connects to a canal 1,200 ft long, dredged to elevation -8 ft, and sized for the flow requirements of two units (Fig. 18). Velocity in the canal at an estimated low level tide of -3.0 is 1.5 fps with a flow of 116 cfs for two units.

Economy is achieved in locating the intake structure adjacent to the power building. A floating log boom placed near the canal inlet prevents debris from entering the intake. Consideration is being given to the installation of permanent facilities for removal of this debris. Water discharges through a single conduit 620 ft long that terminates at the discharge headworks. From this point a canal runs north and returns the water to Humboldt Bay on the other side of Buhne point. Maintenance expense has been confined to periodic dredging of the intake channel and removal of debris collected by the log boom.

SAN DIEGO GAS AND ELECTRIC COMPANY SYSTEM

Three thermal plants with a total capability of 672,000 kw serve San Diego County. A fourth station named South Bay Power Plant is presently (1959) under construction. The first 136,500 kw unit is scheduled for operation in 1960.

Encina Steam Plant.— This station is located on the south shore of Agua Hedionda Lagoon at Carlsbad, 34 miles north of San Diego. Operation of the first unit started in 1954; present capacity consists of 3 units totaling 300,000 kw. The cooling water system is designed to support 600,000 kw. Present flow is 330 cfs, and the ultimate flow will be 600 cfs.

Selection of the cooling water arrangement posed the major problem in development of the site. Numerous schemes were studied. The coastal area was subject to heavy storms and sand movement that made off-shore conduits expensive. The plan adopted (Fig. 19) takes water from the lagoon and returns it to the ocean through an open flume under the highway and a heavily riprapped channel across the beach. The mouth of the lagoon under pre-project conditions was closed by a sand barrier formed by ocean action, which was cut through infrequently when heavy rains caused high flows to the ocean. To provide a tidal prism in the bay large enough to keep the lagoon entrance open more than 4,000,000 cu yd of material were dredged from the lagoon over an area of 240 acres, and to a depth 8 ft below mean sea level. Periodic removal of sand was expected to be necessary within the lagoon and a small dredge was purchased for that purpose. Dredging is done once every two years. Water temperature inside the lagoon varies between 54° F and 76° F, which is usually 2° or 3° above ocean temperature.

Although the land under the surface of the water as well as the south bank is owned by the company, frequent patrol by station personnel is necessary to keep boats and swimmers out. There is considerable public pressure to make the lagoon a boat harbor. The entrance is protected by a log boom that small boats have been known to cross. From the intake structure water is conveyed 520 ft to the station through two 8 ft by 4 ft tunnels. These are cleaned manually once each year. An electric fish screen at the entrance assists in preventing entrance of fish and, during 4 yr of operation, has presented no particular problem. The 13° F rise in temperature through the plant has not interfered with the growth of fish in the discharge channel. Trash removed from the traveling screens is returned to the discharge tunnel.

Silver Gate Station.—This plant is located on the waterfront of San Diego Bay, in an industrial area. Initial operating date was January, 1943. The plant

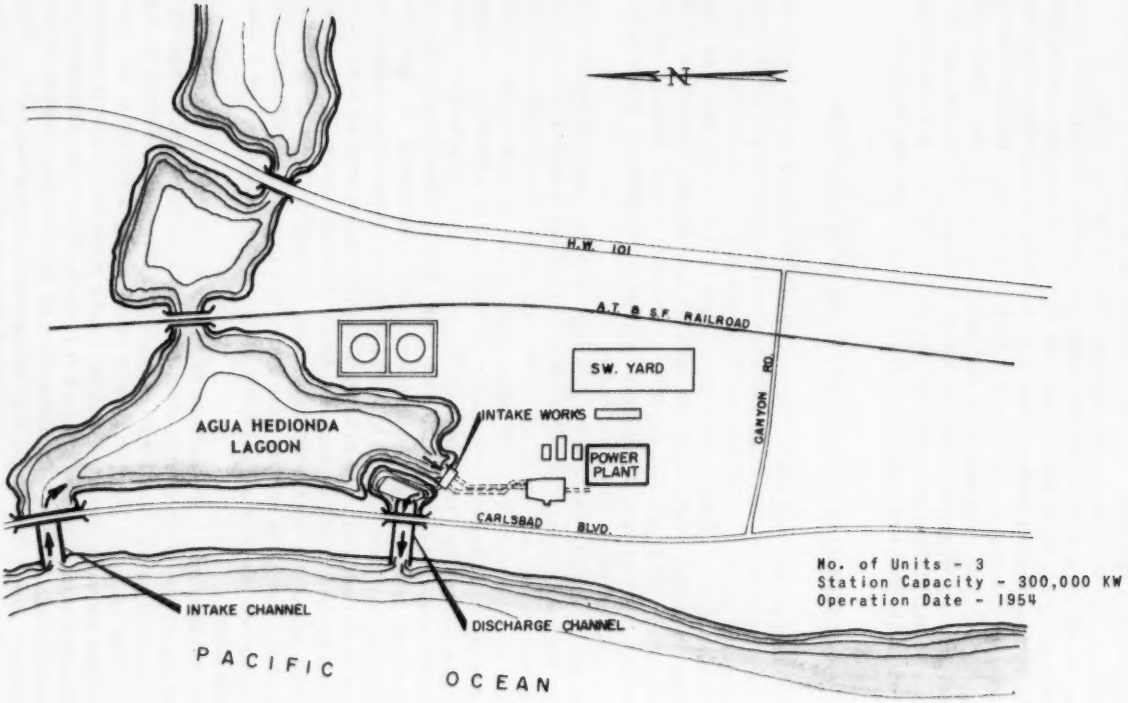

FIG. 19.—ENCINA STEAM STATION COOLING WATER SYSTEM

has four units, with a total capacity 215,000 kw. The cooling water flow is 333 cfs. Water is conveyed to the station through two 8 ft by 8 ft tunnels 760 ft long, and returned to the bay through two tunnels of the same dimensions. This duplicate arrangement permits taking any tunnel out of service for cleaning and inspection without affecting station capacity.

Of particular interest is the relative location of the tunnel inlet and discharge openings (Fig. 20). Separation between these openings is about 200 ft. Steel sheet piling between these two points, and extending about 100 ft from the shoreline, has proved sufficient to prevent recirculation. During the early years of operation fouling of the intake tunnels by marine growth was extensive. However as a result of pollution of San Diego Bay in recent years marine growth has been greatly reduced, and flushing of each tunnel twice a year now maintains reasonable cleanliness. All debris is discharged to the bay. Annual water temperature ranges from 61° F to 74° F. This cooling water arrangement has proved to be a simple, low cost, but effective installation.

DEPARTMENT OF WATER AND POWER, CITY OF LOS ANGELES SYSTEM

The power system includes four thermal plants, three of which are coastal plants with a capability of 576,000 kw. A new station named the Haynes Plant and situated about one mile inland from Alamitos Bay is in the early stages of design (as of 1959). Cooling water will be drawn from a pleasure boat harbor located about a mile from the site.

The most recent addition to this system is the Scattergood Plant located on Santa Monica Bay near El Segundo. Cooling water is conducted to the plant through long submarine pipelines extending into the ocean and resting on the ocean floor.

A more detailed description of the Scattergood and Haynes cooling water arrangements has been given by L. T. Mariner and W. A. Hunsucker.[6] They point out that although these are both coastal plants the site locations presented design problems of an entirely different nature. The Haynes cooling water system is much longer and involves crossing under the San Gabriel River. Nevertheless capitol costs are expected to compare favorably with the submarine facilities at Scattergood. It is concluded that a more remote site using an improved harbor as a source of cooling water can prove more economical than a plant site directly on the ocean front.

Seal Beach.—This station is situated on the ocean front at the mouth of the San Gabriel river in the city of Seal Beach. Two units of 37,500 kw each were installed in 1925 and 1928 respectively.

The original cooling water intake structure was located on the river bank adjacent to the station. Tidal conditions created a sand bar in the river at the entrance to the intake (Fig. 21). Cooling water flow to the station became seriously restricted and caused plant outages on several occasions. Dredging became a continuous operation. The river channel was clear only during the infrequent periods of heavy run-off. It became evident that the source of cooling water should be relocated to an area substantially free from sand movement. A levee was constructed on the opposite side of the river at the entrance to Alamitos Bay. From the original intake structure two 8 ft-diam conduits extend under the river bed, penetrate the levee and terminate in a new intake

[6] "Ocean Cooling Water Systems for Two Thermal Plants," by L. T. Mariner and W. A. Hunsucker, Proceedings, ASCE, Vol. 85, No. PO 4, 1959.

FIG. 20.—SILVER GATE STEAM STATION

FIG. 21.—SEAL BEACH STEAM STATION

structure on Alamitos Bay (Fig. 22). The levee provides an area of comparatively calm water for the intake and forms an effective barrier against sand movement in the river; discharge is to the San Gabriel river.

Harbor.—The station is situated at the Los Angeles Harbor in Wilmington and consists of five units with a total capacity of 355,000 kw. The first unit was placed in service in 1943. Cooling water is taken from slip No. 5 and flows through two 8 ft-diam conduits 1,355 ft long to the screen and pump structure at the station (Fig. 23). Discharge is to the West Basin.

Water pollution, prevalent in the area, is caused by adjacent fish canneries and from ships docked in the harbor. The intake conduits were designed originally for 550 cfs. Later additions of larger units increased the flow to 700 cfs. Drawdown, produced in the intake tunnels during operation with all pumps in service and low tide conditions, can approach minimum pump submergence requirements, particularly if fouling of tunnels becomes acute. Tunnel cleaning is normally performed three times each year.

CONCLUSIONS

The general review of the cooling water systems of the 18 steam stations briefly summarized in this paper leads to the following comments:

1. Although the availability of cooling water has been a major factor in the selection of plant sites, the designer is usually faced with making a choice of several alternate cooling water arrangements after the site is purchased. Economic considerations usually prevail, but they are not necessarily paramount. Public relations and preservation of fish life and other natural resources must also be recognized.
2. There has been a steady improvement in the design of cooling water systems over the past 30 yr as a result of a better understanding of operating and maintenance problems.
3. Desirable station sites are becoming difficult to obtain in many areas, thus emphasizing the need for maximum utilization of existing and future sites.
4. A variety of methods have been used successfully in meeting operating problems connected with marine growth, fish preservation and sand deposition. Thermal shock treatment has become a standard procedure for control of mussels and other marine organisms. The fish problem has been greatly minimized by trapping at intakes and screen wells, and by the installation of velocity caps which change the direction of flow entering submerged pipes from vertical to horizontal. The advice of consultants or specialists in the field of marine biology assisted materially in this work. In general each design is tailored to meet the local conditions economically.
5. Problems associated with open ocean intakes as distinguished from intakes and sheltered bays are mainly the following: a) Sand movement or unstable bottom; b) Marine growth control; and c) Problems associated with the destruction of fish or clogging of screens by fish.

The sand problem is a complicated one along the California coast. The quantity of shifting sand varies tremendously with location. Near the Edison company's Mandalay plant it is estimated to be in the order of magnitude of 1,200,000 cu yd per year. Sites to the southeast or downcoast from large stream will be affected by heavy sand travel for a period of years following a large flood. Rapidly eroding coastlines may also contribute large amounts of

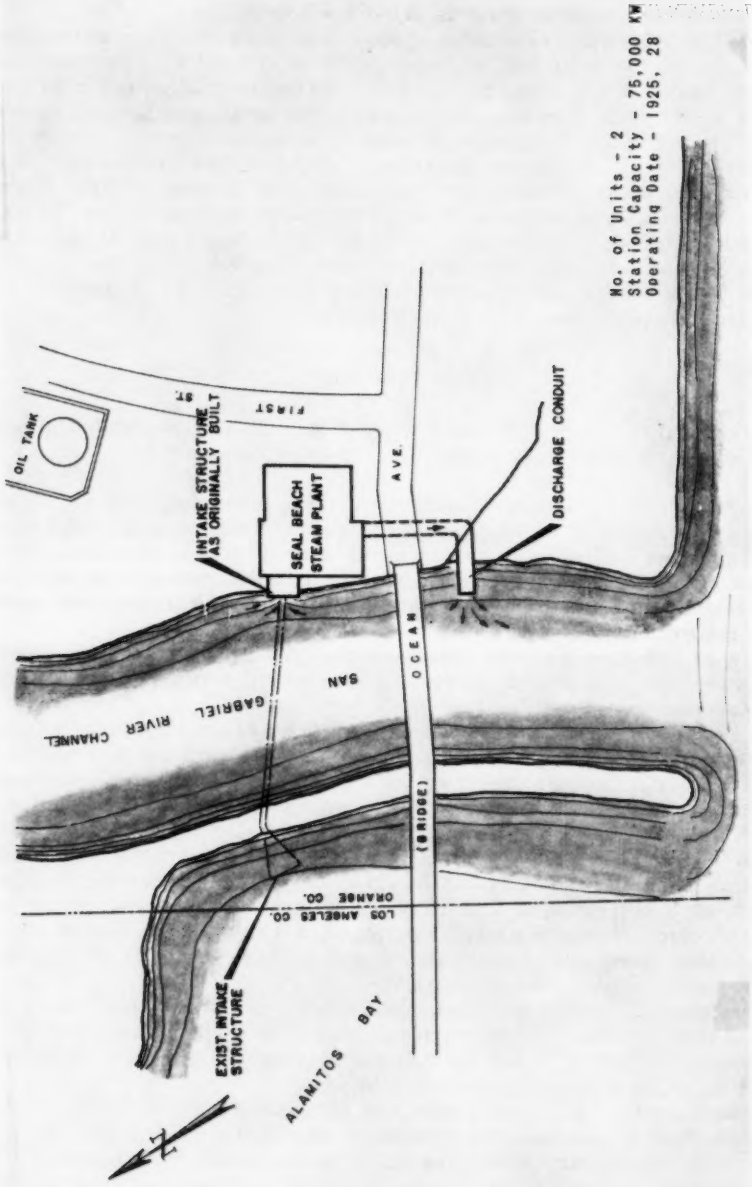

FIG. 22.—SEAL BEACH STEAM STATION COOLING WATER SYSTEM

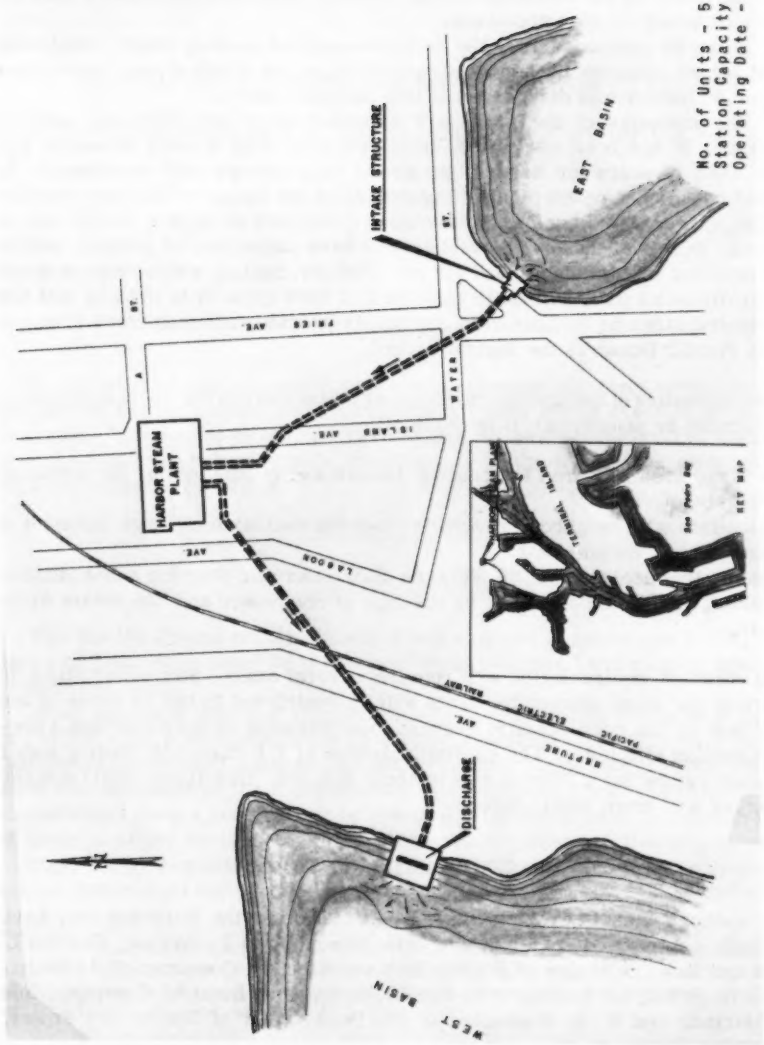

FIG. 23.—HARBOR STEAM STATION COOLING WATER SYSTEM

sand. At some locations a submarine canyon close to shore acts as a permanent trap to bleed off sand accumulations to deep water, and intakes down-coast from such submarine canyons or rocky promontories are apt to have little trouble with sand. The sand problem is not confined to the surf zone but may extend off shore to water 30 or 40 ft deep.

Construction of groins, training dikes, offshore breakwaters and harbor entrance facilities may have drastic effects on sand problems at intakes depending on the exact relation of the intake to the facility. Because of the complex nature of sand problems it is wise economy to retain consultants who are experts on beach control problems.

6. It can be concluded that the most economical cooling water installation for the larger stations presently in operation is one which draws water from a protected harbor and discharges it into the open ocean.

7. The problems of the future are expected to be of a different order of magnitude. It has been estimated, conservatively, that energy demands will double each 10 years for several decades. This energy will necessarily be supplied by thermal power plants. Regardless of the nature of the fuel, whether it is gas, oil, coal or uranium, tremendous quantities of cooling water will be required. Plants of the future are likely to have capacities of several million kw. Assuming a water use of 1.2 cfs per 1000 kw, cooling water requirements for a 5 million kw plant will be in the order of 6000 cfps. It is evident that few if any natural streams in California can safely provide such quantities of water, and the Pacific Ocean is the logical source.

The alternative of using evaporative type cooling towers for such large plants will probably be uneconomical because of:

a) The large area required for cooling towers and to provide for the optimum arrangement.
b) Difficulties with neighbors resulting from the emission of large volumes of vapor or high humidity air.
c) Consumptive use of about 60,000 acre-ft of water per year for a five million kw plant, modified somewhat by the type of condenser and the steam cycle used.

The effect of recirculation will require careful study. Salt water sites, in California, for these mammoth plants will be restricted to two or three of the larger bays or the open ocean to prevent local warming of the water and a lowering of station efficiency. The capitalized value of 1° F change in cooling water inlet temperature for a 200 mw unit is about $25,000. This figure will increase with larger and more costly units.

ACKNOWLEDGMENTS

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CONTAINMENT STUDIES FOR AN ATOMIC POWER PLANT[a]

By William McGuire,[1] M. ASCE, and Gordon P. Fisher,[2] F. ASCE

SYNOPSIS

A summary of the structural aspects of insuring complete containment for the Enrico Fermi Atomic Power Plant in the improbable event of an accident is given. The nature of the critical loading, as well as methods of investigation of structural behavior as a consequence of the loading, is discussed for such emergency conditions.

INTRODUCTION

The Enrico Fermi Atomic Power Plant is under construction (1959) on the shore of Lake Erie about 30 miles southwest of Detroit and about 6 miles east of Monroe, Mich.

In all stages of planning and design, safety of operation has been a primary criterion. All possible sources of energy-releasing accidents have been exhaustively studied and positive preventive measures have been taken which reduce the likelihood of their occurrence to essentially zero. However, the designers have gone a step further by assuming simultaneous or sequential failure of several safety devices resulting in the rapid release of thermal or nuclear energy, and by requiring that the products of such accidents be absolutely contained within the reactor building. Thus, the primary concern of containment

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[a] Presented at the October 1959 ASCE Convention in Washington, D. C.

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analysis is not with the performance of the structure during normal operation, but with its behavior under these emergency conditions. This paper is a summary of some of the containment problems, the methods used to investigate them, and the steps taken to ensure the integrity of the structure in the highly improbable event of an accident.

Although all of the atomic power plants that have been erected in this country in the past decade have been designed to meet exacting standards of reliability and safety, there has not been formulated, as yet, any general method or approach for studying containment problems. This has led inevitably to the overdesign of containment structures in the interests of safety. It is hoped that as more evidence on the nature of nuclear excursions and structural response to dynamic loads is accumulated, more realistic and economical design procedures can be developed.

DESCRIPTION OF THE FACILITY

A brief description of the reactor, shielding, and containment features is necessary for an understanding of the containment problems. Complete descriptions of the plant are available elsewhere (2,3,4).[3]

Fig. 1 is a sectional elevation of the reactor plant. Liquid sodium is pumped into the bottom of the reactor, flows upward through the core and blanket, and is carried off to an intermediate heat exchanger from which it returns to the sodium pump. There are three identical closed loops of this type. Secondary sodium loops conduct heat from the heat exchangers to steam generators located outside the containment vessel. Power is generated in a conventional turbo-generator of 150,000 kw rated capacity located in a separate building.

The core of the reactor, containing U-235 and U-238 fuel alloy elements and control rods, approximates a cylinder 31 in. in diameter and height. Surrounding this are blanket elements containing U-238 alloy. These, in turn, are surrounded by approximately 12 in. of stainless steel in the form of removable rods, laminated plates, and the lower reactor vessel wall. The main function of the stainless steel is to serve as a thermal and radiation shield.

The reactor vessel is divided into three compartments: the lower reactor vessel, the upper reactor vessel, and the transfer rotor container (Fig. 1). The stainless steel upper vessel, which contains fuel handling, positioning and control equipment, is about 14 ft in diameter and 1-1/2 in. thick and lined with 3-1/2 in. of stainless steel plate. At its top is a circular rotating plug about 12 ft high and about 9 ft in diameter. It is made of laminations of stainless steel, borated graphite, cast boron steel, carbon steel, and stainless steel wool, and weighs about 110 tons bare or 140 tons with all accessories. The plug rests on a ring of ball bearings on the reactor walls but is not anchored to the vessel. The transfer rotor container, which is the transfer area between the external and internal fuel handling systems, is offset from the lower reactor vessel. The entire reactor vessel is supported at the transition between the upper and lower vessels by steel struts.

Surrounding the reactor vessel is the primary shield tank, a 5/8-in. thick carbon steel cylinder with a maximum diameter of 24 ft. It is surmounted by a dome about 18 ft in diameter and 15 ft high. All the space between the primary shield tank and the reactor vessel not occupied by structures or piping,

[3] Numerals in parentheses, thus (1), refer to corresponding items in the Bibliography (see Appendix).

is filled with clay brick, borated graphite, ordinary graphite or carbon, with an average density of 100 lb per cu ft. The total weight of this shielding material is approximately 800,000 lb.

The secondary shield wall (Figs. 1, 2, 3) is composed of intrusion, prepacked concrete with a minimum strength of 3,000 psi. Each face of the wall is completely covered with 1/2-in. carbon steel plate, fully butt welded at all joints. This serves as formwork for the concrete and is firmly anchored to it by transverse tie bars spaced 21 in. on center in each direction. The thickness of concrete varies from 31 in. to 39 in. The effective height of the wall is about 31 ft.

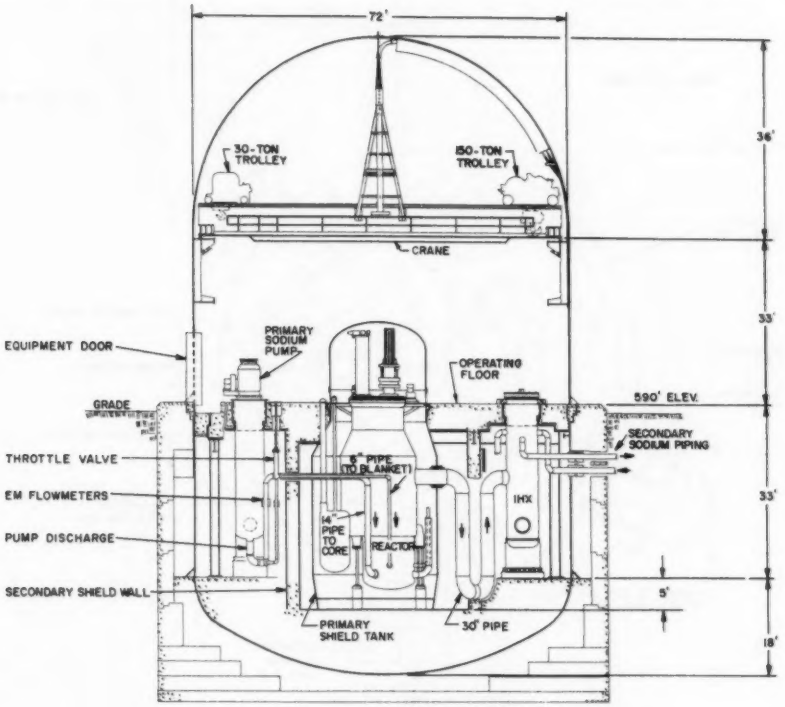


FIG. 1.—ELEVATION OF REACTOR PLANT

The distance from the reactor core center to the closest point on the inner face of the wall is about 12 ft - 9 in. In addition to the face plates, the wall is reinforced vertically by two layers of No. 7 bars at 6 in. on centers and horizontally by two layers of No. 6 bars at 12 in. on center. Special reinforcement is provided at all openings. There are also thirteen 14 WF 167 columns embedded in the wall for the support of the operating floor above.

The operating floor, also heavily reinforced and about 5 ft thick, is primarily a biological shield. The top 1/4-in. is of steel plate. On the bottom there is a minimum of 3-1/2 in. of steel plate. The floor is supported by a network of

steel beams spanning between the columns embedded in the secondary shield wall and an outer row of seventeen 14 WF 167 columns.

The containment vessel is a vertical cylinder 72 ft in diameter with a hemispherical head and hemiellipsoidal bottom. Its overall height is 120 ft. It is of all-welded construction using the American Society of Testing Materials (ASTM) A - 201 B steel furnished to meet the requirements of ASTM A - 300, which provides for a minimum Charpy impact resistance of 15 ft-lb at -50° F. Plates are normalized and aluminum-killed. Design is in accordance with the American Society of Mechanical Engineers (ASME) Code for Unfired Pressure

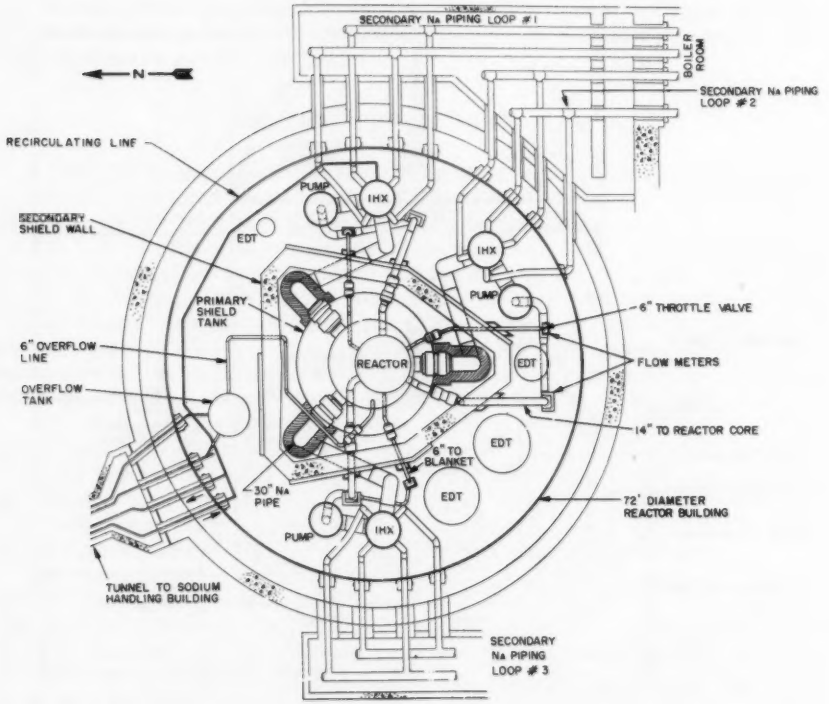
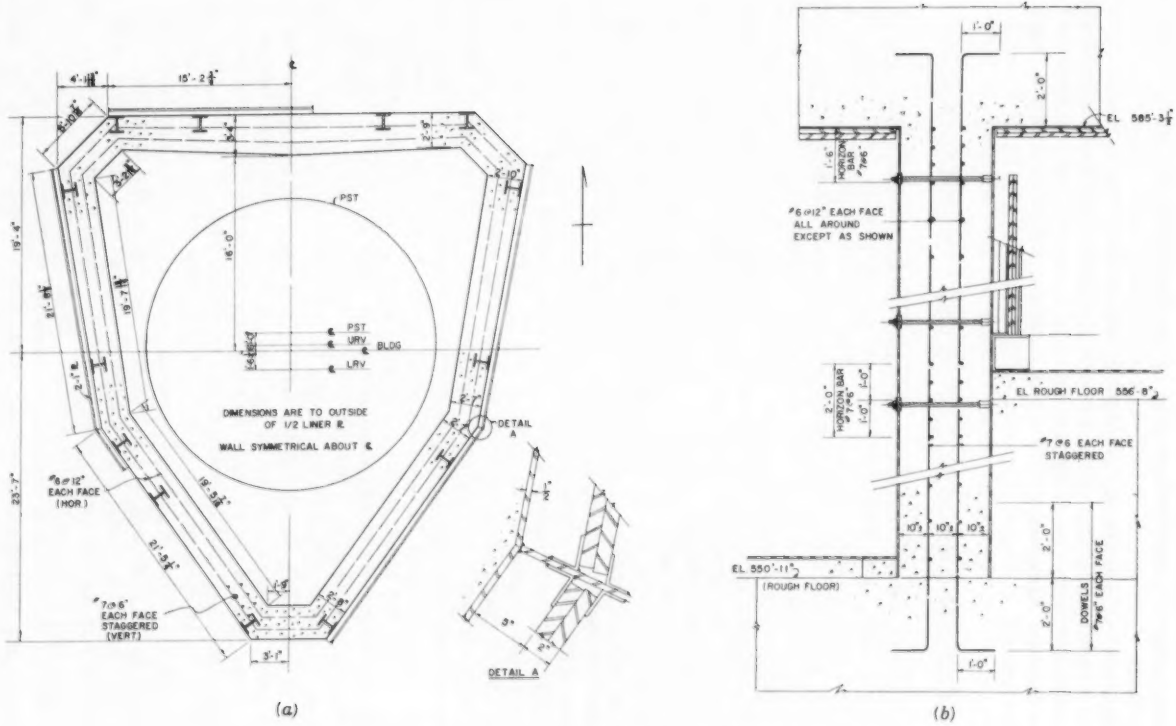


FIG. 2.—PLAN VIEW OF REACTOR PLANT BELOW OPERATING FLOOR

Vessels, Section VIII, and the vessel has been so stamped. Full penetration butt welds are used in the plate. The welds have been radiographed and the entire vessel has been subjected to a 40 psi pressure test as required by the ASME Code. Most of the plates in the cylinder and bottom are 1.03 in. thick and 0.52 in. in the dome. The vessel was designed conventionally as a pressure vessel for 32 psig internal pressure and for 2 psig external pressure plus normal wind and snow loads. It was also designed to support a 150 ton crane riding on circular girders welded to the shell. The lower hemiellipsoidal section is surrounded inside and out by concrete which provides continuous support down to the underlying rock.



(a) (b)

FIG. 3.—SECTIONS THROUGH SECONDARY SHIELD WALL

There is a 13 ft - 6 in. high by 11 ft - 3 in. wide equipment door in the shell above the operating floor, as well as two air locks, one 3 ft by 6 ft, the other 3 ft in diameter. There are numerous other small penetrations above and below the floor.

In addition to the stiffening provided by the crane girders, there is a lower internal stiffening ring just below the operating floor. Surrounding the containment vessel below the operating floor level is a plain concrete biological shield with a minimum thickness of 3 ft - 0 in. This is isolated from the vessel by shielded joints.

NATURE AND MAGNITUDE OF ASSUMED EMERGENCY LOADS

Two types of possible accidents were postulated for the Fermi plant by reactor physicists: a sodium fire and a nuclear excursion. Extensive calculations and tests by the Atomic Power Development Assoc. Inc. (APDA) have shown that a sodium fire would result in a maximum possible pressure of 28 psig within the containment vessel and a shell temperature of not more than 460° F. Furthermore, this temperature would not be reached until about 24 hr after the initiating incident. This presents a conventional problem in pressure vessel design. The hypothetical nuclear excursion, although the result of atomic fission, could not remotely approach the energy release of an atomic weapon because there is no way to maintain a critical configuration of fuel long enough to cause this to occur, nor is the assembly rate sufficiently rapid. In fact, any accidental energy release of this kind is based on a set of highly improbable premises that make the possibility of its occurrence extremely remote. Within the limits of such assumptions, APDA physicists have estimated that the maximum credible energy release would be less than that of 1,000 lb of TNT, or if more reasonably based, of the order of 300 lb of TNT. It was also determined that the rate of energy release from an excursion would be of the same order of magnitude as that from ordinary high explosives such as TNT or Pentolite.

Since excursions of this order would cause irreparable damage to the reactor, it is not necessary to keep stresses in the supporting and surrounding structure below the conventional allowable values. Rather, the problem is to investigate all possible modes of failure and to make sure that either the ultimate capacity of each part of the structure is not reached, or that any internal structural failure is not serious enough to cause breaching of the outer containment vessel. Thus, the analytical approach is logically one of seeking reasonable upper limits of loading on each structural element and making sure that the lower limit of the element's resistance to this loading is not exceeded. The major considerations in containment analysis, wherein lie the most serious difficulties of decision, are 1) the methods from structural mechanics that most reasonably define structural behavior, and 2) the nature and magnitude of the loading. In the first instance, substantial theoretical and experimental evidence on the behavior of statically loaded structures near failure has been accumulated only in the last twenty years (since 1940) and reliable analytical procedures are still in the formative stages. Even less quantitative information is available on the failure of structures subjected to dynamic loads. Obviously then, no precise analysis is possible. But, since absolute integrity against rupture of the outer containment vessel must be guaranteed, there is little choice but to use the information and methods presently available and to make conservative estimates of the structural action. In the second instance, there

are no known available empirical data on the blast pressures and impulses resulting from nuclear excursions, so that data for conventional high explosives are resorted to. The blast effects of high explosives in free air are well-established. On the other hand, within a geometrically complex structure containing solid and liquid materials of various energy absorbing characteristics, blast effects are more difficult to predict.

The response of a structure to blast loading can be predicted only if the pressure-time history of the impulse is known. In particular, the air blast on any affected structural element may be defined in terms of the maximum instantaneous overpressure, the duration of the impulse, and the shape of the pressure-time curve. These characteristics and associated scaling laws, which permit their application to structures located at varying distances from the blast center, have been developed and corroborated for high explosives. An important question to be settled before using these data for excursion simulation is whether the rate of energy release is of the same order of magnitude for the high explosive as for the nuclear excursion it is intended to represent. Complete agreement between power-time curves cannot be expected, nor is it necessary for estimating ultimate containment capacity. The essence of the comparison is whether the energy release occurs in a matter of microseconds for one and milliseconds for the other. The critical rate of power increase for high explosives is always very rapid and the majority of the energy is released in microseconds, being largely independent of the quantity of explosive (5). For the postulated nuclear excursions, the major fraction of the energy would be released in about 60 microseconds. This correspondence justifies the use of conventional blast data for the Fermi reactor. In this sense, high explosive data may not be appropriate for use with other reactors.

The best available information on air blast about high explosive charges is considered to be that reported by Hoffman and Mills (6) which gives explicit data on peak pressures, impulses and scaling laws. Briefly, the scaling law that permits their application in containment studies states that a given pressure will occur at a distance from an explosion proportional to the cube root of the energy yield, or specifically, that identical peak pressures will be experienced at the same scaled distance, $Z = R/W^{1/3}$. Thus, general curves of pressure and impulse plotted against scaled distance may be determined from test explosions, and then used to find the pressure and impulse for any other charge-weight and blast-radius. Cube root scaling may be applied with confidence over a wide range of explosion energies (7). Scaling applies equally well to all blast parameters such as maximum pressure, duration of positive pressure phase, time of arrival of shock front, scaled impulse, and so forth.

The maximum pressure increase over ambient pressure experienced by a solid wall as it is struck by a shock wave traveling on a path perpendicular to a wall is known as the peak face-on (or reflected) over-pressure. It may be from two to about eight times the corresponding side-on overpressure, which is the overpressure behind a shock wave of the same strength, but unimpeded by a reflecting surface. As a shock wave reflects from a wall, the face-on overpressure decays rapidly from the peak value as in Fig. 4(a). The positive pressure phase is usually followed by a negative pressure or rarefaction phase of much smaller magnitude.

The pressure decay is so nearly linear and the rise time to peak pressure so nearly instantaneous that it is customary to assume a simple pressure-time relationship, $p = p_o\left(1 - \frac{t}{t_1}\right)$, and zero rise time, as in Fig. 4(b). The small

rarefaction phase is generally neglected. This approximate linear relationship accords well with experimental data available and has the further virtue of being simple mathematically, resulting in response expressions in closed form.

For explosions confined within closed vessels, there exists the possibility that the initial shock wave may reflect from the vessel walls, converge back on the blast center to form another strong shock which then would rediverge

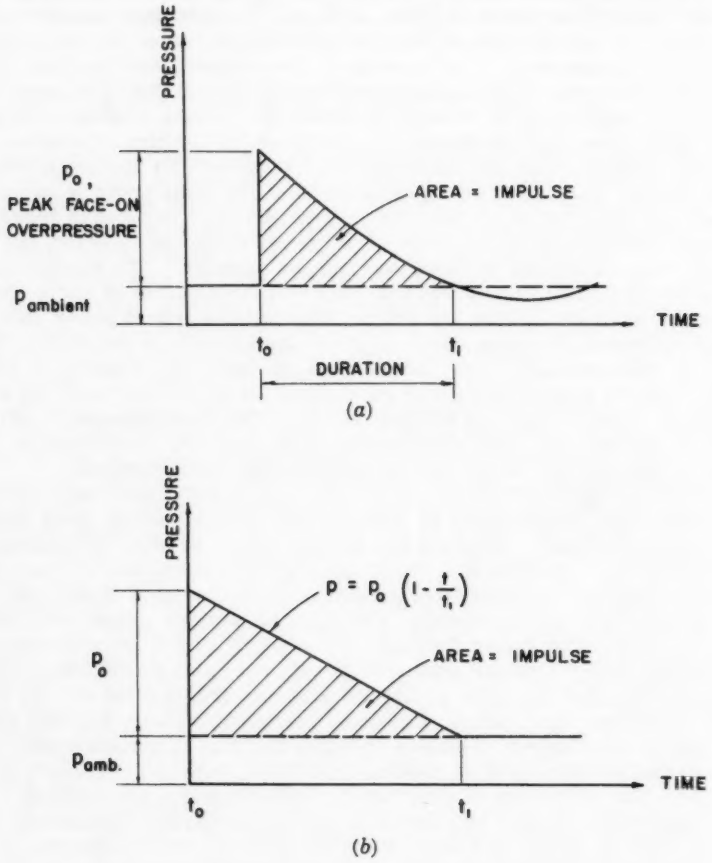


FIG. 4.—PRESSURE-TIME RELATIONSHIPS FOR HIGH-EXPLOSIVE BLAST

to apply a second transient loading to the vessel walls. This sequence may, theoretically, be repeated many times, with successive pressure peaks reducing in magnitude as kinetic energy is converted to other forms. This phenomenon has been observed experimentally by several investigators (5,6,8). However, the possibility of damage from this cause may be discounted for this reactor study on the basis that secondary wave reflections subsequent to the initial shock, if they occur at all, will be much smaller than the initial peak

overpressure and negligible in effect. It is highly unlikely that a reactor core excursion, within several heavy and geometrically irregular containers and shields, could produce secondary shocks in the face of the energy absorbed by such internals and the inability to refocus reflected waves.

An excellent summation of the secondary shock problem has been made by W. E. Baker (9) in which he describes theoretical and experimental studies on shells subjected to internal blast loading. In part, he says:

> "Since there is considerable dissipation of the mechanical energy of the blast wave as heat, the repeated shocks will be repeatedly weaker and weaker, until no discernible shock can be detected. The final result of the process is a static pressure and temperature rise in the shell caused by the addition of the heat of detonation of the explosive to the air within the shell. Some experiments conducted within the (Aberdeen, Md.) 30-ft diameter spherical blast chamber . . . have shown that the initial blast loading of a spherical shell from centrally located explosive charges far exceeds the secondary shocks, and that the static pressure rise is usually insignificant."

The main structural units of the reactor building, such as the secondary shield wall and the outer containment vessel, would not experience the full explosive effects of a postulated TNT equivalent excursion. Energy would be expended in crushing the reactor blanket material and rupturing the reactor vessel, thus reducing that available for producing air blast. Further attenuation of the blast effect would result from energy lost in irreversible conversion to heat, crushing and fragmenting the graphite shielding, and heating and vaporizing the sodium coolant. The problem of estimating such energy losses is under very active investigation but it is a very complicated one. The authors know of no currently available, reliable method for computing these losses with certainty. However, it is certain that the total energy loss is very large. In another study of the Fermi reactor, the Naval Ordnance Laboratory has stated (10):

> ". . . a heavy case such as that surrounding the reactor core reduces the effective weight of an explosive charge to produce air blast by at least 70% compared to a similar uncased charge. Greater blast reduction exceeding that of steel-cased high-explosive weapons can be expected because of the blast absorbing potential of the various kinds of material surrounding the core. Thus an air shock not greater than that which would be produced by a bare 300-lb charge of TNT is generated in the reactor compartment from an explosion of 1,000 lb of TNT at the core."

Other arguments based on energy lost by irreversible heating have been advanced and support the contention that such large attenuation is possible.

Thus, in evaluating the effects of a 1,000-lb TNT equivalent nuclear excursion, the air blast resulting from a 300-lb TNT explosion may be used. (This is not to be confused with the 300-lb TNT equivalent nuclear excursion calculated as the lesser accident in the Fermi reactor. A similar reduction in energy would apply to it.) It must be pointed out that the reduction in air blast is not an unencumbered gift. Some of the energy not available for producing air blast would propel fragments of the ruptured material against the surrounding protective structure. Such missiles, however, would be harmful only if they struck the outer containment vessel with penetrating force, which

is possible only if they first breach the secondary shield wall or operating floor. The secondary shield wall, then, must be investigated for the effects of the attenuated air blast and internal missiles. The containment vessel must be investigated for the effects of an air blast further attenuated by the secondary shield wall and of any missiles which can be shown to reach the shell. Missiles include those from the reactor region that penetrate the secondary shield wall and those created outside the wall by partial fragmentation of the wall itself or of material between the wall and containment vessel. In this respect R. O. Brittan has said (1),

> "There is little possibility of a shock wave being transmitted to the air between the primary structure (secondary shield wall-authors) and the outer container at the shield interface. A shock loading to the outer shell is not considered in design. At best, only a sudden pressure rise to a calculable maximum can be expected, with a pressure-time relation depending on the exit time of the core materials and flashing or burning times."

As a conservative measure, however, the Fermi containment vessel has been studied for the effect of a 300-lb TNT air blast, representing the initial 70% attenuation of the 1,000-lb TNT equivalent excursion and ignoring the presence of the secondary shield wall. The static pressure rise would not be critical. If we assume a constant-volume adiabatic process with all the energy of detonation of 1,000 lb of TNT (4.54×10^8 calories) effective, the pressure rise would be of the order of 13 psi, much less than the 32 psig for which the vessel was designed.

Reasoning from the foregoing discussion, the following initial guiding principles were adopted for the analysis:

(1) That an excursion of this reactor may be suitably simulated by a high explosive reaction of equivalent energy of detonation.

(2) That cube root scaling of explosion data is valid.

(3) That the positive pressure phase of the blast has a triangular pressure-time relationship.

(4) That rarefaction and secondary shocks may be ignored.

(5) That detonation energy is attenuated by at least 70%. Consequently, the secondary shield wall is considered to experience an air blast equivalent to that of a bare 300-lb TNT charge situated at the reactor center.

(6) That the containment vessel is subjected to the same 300-lb TNT blast, and that the static pressure rise is not critical.

(7) That once containment is assured, under items (5) and (6), consideration must be given to the effects of any internal missiles that may be generated within the reactor building.

SECONDARY SHIELD WALL

Blast Containment.—The monolithic structure composed of the secondary shield wall, the operating floor, and the bottom floor is a highly complex unit whose structural action must be greatly simplified for analysis, and in such a way as to ensure a conservative estimate of damage. The walls and the floors were treated separately.

Dynamic analysis was necessary in view of the large inelastic deformations resulting from high blast pressures. Static analysis, using "equivalent static

loads" to replace dynamic loads, is inadmissable since such loads applied for an indefinite time may incorrently indicate arbitrarily large deformations. On the other hand, large loads applied for brief intervals and resisted by inertial accelerations of the structure may result in deformations which, though large, will not rupture the structure, perhaps even leaving it in serviceable condition.

A shock wave, radially propagated from an explosion at the center of the reactor and impinging on the wall, may be visualized as being resisted by interaction of two major types of elements:

1. Vertical strips of the sidewall acting as beams (or one-way slabs) supported at top and bottom.
2. Polygonal hoops or frames as cut by horizontal planes through the sidewalls.

Since these elements act together to carry the blast loading, the capacity of either type alone will be less than the combined capacity; thus, if either type of element can carry by itself the stipulated blast pressure, it may be concluded that the combined action will support a higher blast pressure.

Primary consideration was given to the vertical elements of the wall. They are stiffer than the polygonal hoops and consequently will carry the major portion of the loading and more nearly govern the overall failure mode of the wall. The sidewall was analyzed on the basis of a typical unit strip of a one-way slab, simply supported at top and bottom over a span of nominally 31 ft. The loading on this unit vertical strip was assumed as a uniformly distributed pressure based on the shortest distance from the core to the side-wall.[4]

The vertical strips were analyzed in accordance with the general theory of rigid-plastic beams, which is based on the development of plastic zones at points of maximum stress (11,12,13). Although this theory is well established and thoroughly covered in the literature, a brief review of the basic theory will be included.

Consider a statically applied uniform load on a simply-supported beam, as in Fig. 5. Assume that it is made of mild steel with an idealized stress-strain diagram (tension or compression) as in Fig. 6. The maximum bending moment in the beam occurs at midspan where, under increasing load, the midspan section finally becomes completely plastified. The value of M_{max} when full plastification occurs is known as the "plastic moment capacity," M_p, at which point the beam sags under load and exhibits an abrupt change in curvature at midspan as a result of the large local plastic deformation. Elsewhere the curvature is relatively small, because it is elastic. In essence, then, the beam acts

---

4 Consideration of non-uniform pressure distribution along the vertical beam is not warranted in view of the other simplifying assumptions of the analysis. In fact, the actual distribution cannot be determined theoretically with any certainty. Usually, the ends of a vertical strip, being at greater radius from the explosion and not at normal incidence with the shock wave, would experience a smaller pressure than the center of the strip. How much smaller is not known, since "the complete analytical solution of even such a relatively simple problem as the behavior of a shock wave incident on a wall at an oblique angle has never been obtained for all angles." On the other hand, shock concentration or focusing in the corners formed by the sidewall with top and bottom slabs increases the pressure in that vicinity and raises it perhaps to the magnitude of the midspan pressure. For a shock running into a 90° corner, "there is a local increase in pressure in the corner of at least four times the initial overpressure in the shock." (Quotations from "The Effects of Atomic Weapons," A.E.C., 1950.) Therefore, an assumption of uniform pressure distribution seems reasonable for the purposes of this study.

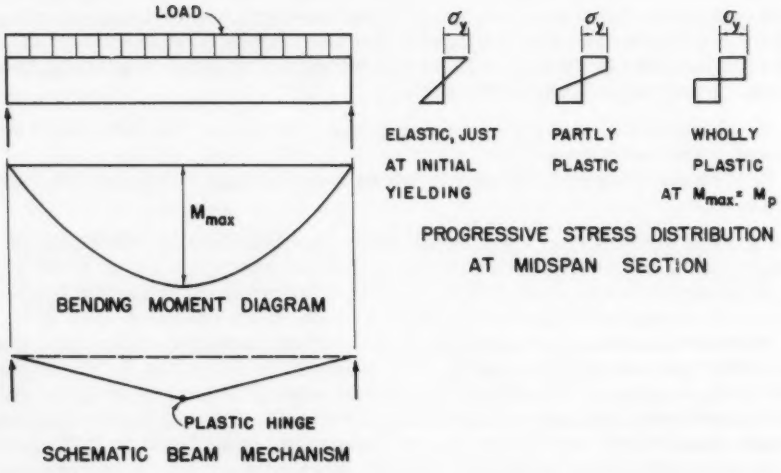


FIG. 5.—PLASTIFICATION OF A SIMPLE-SUPPORTED STEEL BEAM

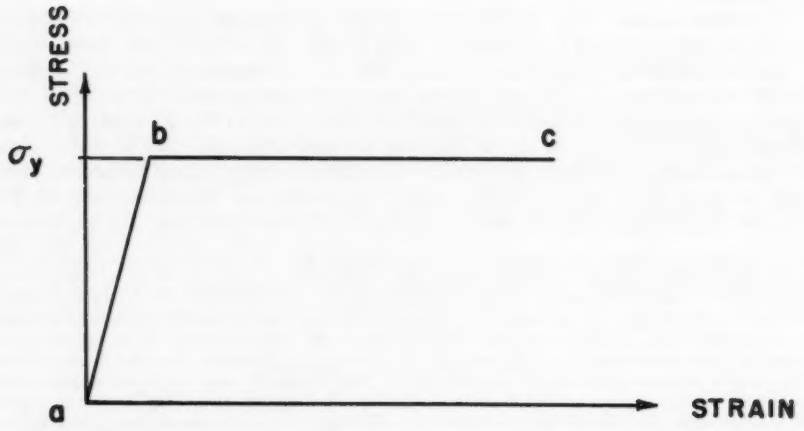


FIG. 6.—IDEALIZED STRESS-STRAIN DIAGRAM FOR STEEL

as two rigid halves connected at midspan by a hinge that allows free rotation once M_{max} has reached M_p. At that point, the beam deflects without limit with no further increase of load, and becomes an unstable mechanism. In addition to the case of plastic bending described above, two other cases were considered, employing a similar concept of plastic sliding (or shear) failure at the supports where maximum shear occurs (12). Plastic sliding was considered alone and combined with bending. The controlling failure mode is governed by the relative plastic resistance in sliding and bending and by the ratio of load to resistance.

The plastic-moment capacity of the typical beam strip was based on an effective cross section consisting of only the steel surface plates unaided by the concrete. The flexural resistance of the concrete was discounted in view of the axial tension in the sidewall and the likelihood of the concrete being cracked and inactive in resisting bending. The axial tension occurs as a result of the blast loading on the top and bottom floors. However, the basic integrity of the concrete, assured by its internal reinforcing bars, was counted upon to maintain the spacing of the surface plates and their beam action. The plastic sliding capacity of the beam was based on the reinforcing steel only, acting in pure tension at its yield point. This assumes that it is possible for the reinforcing bars to bend locally at the beam ends and become more or less aligned with the shearing force, resisting the shear at the supports by tension in the bars. The steel surface plates, having insufficient embedment or attachment, were not counted upon for end resistance. In effect, then, the wall was considered as though it were a beam in which the flanges (steel plates) resist all of the bending and the web (reinforcing bars, concrete, etc.) carries the end reaction only.

Under a dynamic loading of large magnitude the beam first deforms elastically and then shortly thereafter develops plastic "hinges" or "slides" leading to a collapse mechanism. A constant load would cause arbitrarily large deformations and collapse once M_p had been exceeded. However, the blast loading decreases rapidly to zero and is continuously resisted by inertial forces developed in accelerating the beam and by plastic resistances, so that beam motion eventually stops. It was the purpose of the analysis to determine whether or not motion stops short of complete collapse of the beam and to predict the order of magnitude of the permanent deformation when the beam comes to rest. Some simplification of the theory may be achieved by neglecting the elastic deformation and assuming that the plastic zones of the beam exist before and during the complete blast pulse.

It was found that failure by simple plastic sliding at the supports is inadmissible for the relative sliding and bending resistances provided. On the assumption of sliding motion only, the maximum bending moment was found to exceed the plastic moment capacity, which indicates that a plastic hinge would form and that the assumed failure mode is not valid. It was also found that the assumption of a plastic hinge at midspan and non-sliding supports could not be realized.

The governing mode of behavior may be shown to be a combination of plastic hinge at midspan and plastic sliding at the supports. A summary of the calculations for this mode follows.

A beam strip such as that investigated has an infinite number of degrees of freedom. However, by assuming complete rigidity between plastic zones and replacing the actual mass of the beam by an equivalent mass, the equations of

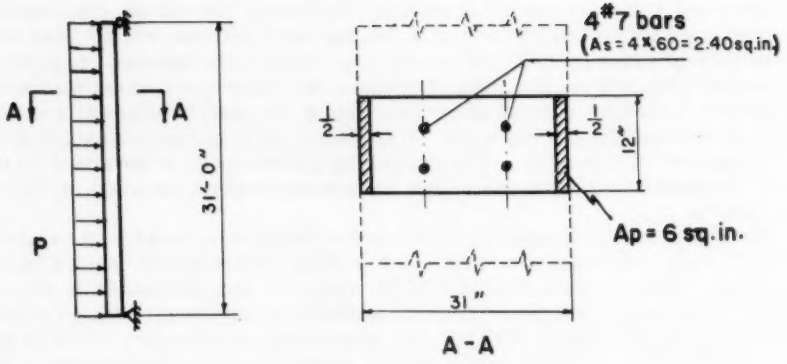


FIG. 7.—BEAM STRIP

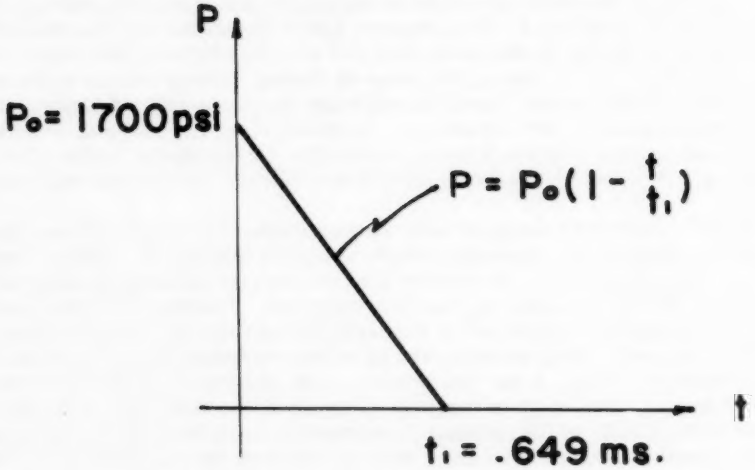


FIG. 8.—LOADING FUNCTION, 300 POUND TNT BLAST

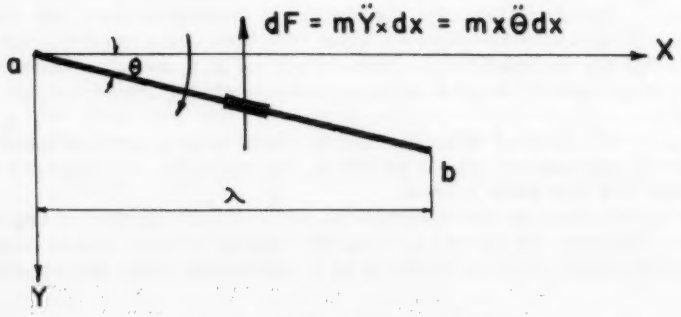


FIG. 9.—ROTATING ELEMENT

motion for a one degree of freedom system may be used. This procedure has been followed and corroborated by a number of investigators.

Fig. 7 shows the beam strip section and Fig. 8 the loading function for a 300-lb TNT air blast 12 ft by 9 in. from the explosion center, which represents the effect of the attenuated 1,000-lb TNT equivalent excursion on the secondary shield wall. This is developed from the data presented by A. J. Hoffman and S. M. Mills (6).

Assuming σ_y = 30,000 psi,[5] the plastic moment and shear capacities are

$$M_p = A_p \ \sigma_y \quad 30.5 = 6 \times 30 \times 30.5 = 5490 \text{ in.-kips} \quad \ldots\ldots (1)$$

$$V_p = A_s \ \sigma_y = 2.40 \times 30 = 72 \text{ kips} \quad \ldots\ldots\ldots\ldots (2)$$

The beam weight is 415 lb per linear ft, and its unit mass, m = 415/32.2 = 12.89 lb sec^2 per ft^2.

The general forces due to rotary inertia of a rotating strip of length λ (Fig. 9) are

$$M_a = \int_o^\lambda x \, dF = m \ddot{\theta} \int_o^\lambda x^2 \, dx = \frac{m \lambda^3}{3} \ddot{\theta} \quad \ldots\ldots\ldots (3)$$

$$M_b = \int_o^\lambda (\lambda - x) \, dF = \frac{m \lambda^3}{6} \ddot{\theta} \quad \ldots\ldots\ldots (4)$$

$$F = \int_o^\lambda dF = m \ddot{\theta} \int_o^\lambda x \, dx = \frac{m \lambda^2}{2} \ddot{\theta} \quad \ldots\ldots\ldots (5)$$

These will be used in establishing the equations of equilibrium of the actual beam strip.

Applying Newton's second law of motion to the free body diagram representing the assumed failure mode (Fig. 10) and using Eqs. 3 through 5

$$\Sigma V = 0 = V_p - \frac{p L}{2} + \frac{m L}{2} \ddot{y}_a + \frac{m L^2}{8} \ddot{\theta} \quad \ldots\ldots\ldots (6)$$

$$\Sigma M_a = 0 = M_p - \frac{p L^2}{8} + \frac{m L^2}{8} \ddot{y}_a + \frac{m L^3}{24} \ddot{\theta} \quad \ldots\ldots\ldots (7)$$

Solving for $\ddot{y}_a$ and $\ddot{\theta}$ leaves

$$\ddot{y}_a = \frac{p}{m} + \frac{24 M_p}{m L^2} - \frac{8 V_p}{m L} = \frac{1}{m} \left(p - q_a\right) \quad \ldots\ldots\ldots (8)$$

$$\ddot{\theta} = \frac{24 V_p}{m L^2} - \frac{96 M_p}{m L^3} \quad \ldots\ldots\ldots\ldots\ldots (9)$$

---

[5] The specified yield strength of intermediate grade reinforcing steel is 40,000 psi, but 30,000 psi was used as a conservative measure in studying the secondary shield wall.

In addition,

$$\ddot{y}_c = \ddot{y}_a + \frac{L}{2}\ddot{\theta} = \frac{p}{m} - \frac{24\ M_p}{m\ L^2} + \frac{4\ V_p}{m\ L} = \frac{1}{m}\left(p - q_c\right) \quad \ldots\ldots (10)$$

in which

$$q_a = \left(-\frac{24\ M_p}{L^2} + \frac{8\ V_p}{L}\right) \quad \ldots\ldots\ldots\ldots (11)$$

and

$$q_c = \left(\frac{24\ M_p}{L^2} - \frac{4\ V_p}{L}\right) \quad \ldots\ldots\ldots\ldots (12)$$

Eqs. 8 and 10 are ordinary differential equations in which p represents the driving force (loading function) as defined in Fig. 8, and q represents the structural resistance as defined in Eqs. 11 and 12.

In general terms, the solution of the equation

$$\ddot{y} = \frac{1}{m}\ (p - q) \quad \ldots\ldots\ldots\ldots\ldots (13)$$

is, for $0 \leq t \leq t_1$

$$\dot{y} = \frac{t}{m}\left[p_o\left(1 - \frac{t}{2\ t_1}\right) - q\right] \quad \ldots\ldots\ldots\ldots (14)$$

$$y = \frac{t^2}{2\ m}\left[p_o\left(1 - \frac{t}{3\ t_1}\right) - q\right] \quad \ldots\ldots\ldots\ldots (15)$$

and, for $t \geq t_1$

$$\ddot{y} = -\frac{q}{m} \quad \ldots\ldots\ldots\ldots\ldots (16)$$

$$\dot{y} = \frac{1}{m}\left[\frac{p_o\ t_1}{2} - q\ t\right] \quad \ldots\ldots\ldots\ldots (17)$$

$$y = \frac{1}{2\ m}\left[p_o\ t_1\left(t - \frac{t_1}{3}\right) - q\ t^2\right] \quad \ldots\ldots\ldots\ldots (18)$$

If the loading and resistance are such that outward motion stops prior to the time t_1, then t_{max}, the time at which this occurs, can be determined by setting the velocity Eq. 14 equal to zero.

$$t_{max} = 2\ t_1\left[1 - \frac{q}{p_o}\right] \quad \ldots\ldots\ldots\ldots (19)$$

which is valid only for $t_{max} \leq t_1$, or when

$$\frac{q}{p_o} \geq \frac{1}{2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (20)$$

The total deflection corresponding to t_{max} is

$$y_{max} = \frac{t^2_{max}}{6\ m} \left(p_o - q\right) \quad \ldots\ldots\ldots\ldots (21)$$

If, on the other hand, $q/p_o \leq 1/2$, then outward motion stops at a time greater than t_1, namely

$$t_{max} = \frac{p_o\ t_1}{2\ q} \quad \ldots\ldots\ldots\ldots\ldots (22)$$

and the total deflection is

$$y_{max} = \frac{p_o\ t_1^2}{2\ m} \left[\frac{p_o}{4\ q} - \frac{1}{3}\right] \quad \ldots\ldots\ldots\ldots (23)$$

By substitution of the following values: M_p = 5490 in.-kips, V_p = 72 kips, L = 31 ft and p_o = 1700 psi, into Eqs. 11, 12 and 20, it is determined that q_a = 599 lb per in. and q_c = 177 lb per in. and also that $\frac{q_a}{p_o} + \frac{q_c}{p_o} \ll \frac{1}{2}$. Therefore, $t_{max} > t_1$. From Eq. 23

$$y_{c\ max} = \frac{1700 \times 144 \times 0.649^2 \times 32.2}{2 \times 415 \times 10^6} \left(\frac{20{,}400}{4 \times 177} - \frac{1}{3}\right) = 0.110 \text{ ft} = 1.31 \text{ in.}$$

and

$$y_{a\ max} = 4.0 \times 10^{-3} \left(\frac{20{,}400}{4 \times 599} - \frac{1}{3}\right) = 0.033 \text{ ft} = 0.39 \text{ in.}$$

also

$$y_{net} = y_c - y_a = 0.92 \text{ in.}$$

In order for the above solution to be valid the shear, V_x, and the bending moment, M_x, must be less than the limiting values V_p and M_p at all points in the beam except at the respective plastic zones assumed in the analysis. These conditions may be checked by using the equations for shear and bending moment. From Fig. 10, and using Eqs. 3, 4 and 5

$$V_x = V_p - p\ x + m \times \ddot{y}_a + \frac{m\ x^2}{2}\ \ddot{\theta} \quad \ldots\ldots\ldots\ldots (24)$$

Substituting for $\ddot{y}_a$ and $\ddot{\theta}$ from Eqs. 8 and 9

$$V_x = \left(1 - \frac{2\ x}{L}\right) \left[V_p \left(1 - \frac{6\ x}{L}\right) + 24\ \frac{M_p\ x}{L^2}\right] \quad \ldots\ldots\ldots (25)$$

Similarly,

$$M_x = V_p\ L \left(\frac{x}{L}\right) \left(1 - \frac{2x}{L}\right)^2 + 4\ M_p \left(\frac{x}{L}\right)^2 \left(3 - \frac{4x}{L}\right) \quad \ldots (26)$$

These are valid for any time, t.

From Eq. 25, points of zero shear and maximum bending moment occur at

$$x_1 = L/2 \quad \ldots (27a)$$

$$x_2 = \frac{V_p\ L}{6\left(V_p - 4\frac{M_p}{L}\right)} \quad \ldots (27b)$$

The solution is valid only if $x_2 \geq \frac{L}{2}$; otherwise plastic hinges will form first at points $x_2 < L/2$, thus violating the assumed mode of behavior. The criterion for $x_2 \geq L/2$ may be written as

$$\frac{V_p\ L}{2\ M_p} \leq 3 \quad \ldots (28)$$

For the case being studied, $\frac{V_p\ L}{2\ M_p} = 2.44 < 3$; therefore, the bending moment criterion is satisfied. A similar shear criterion is also satisfied and the assumed mode of behavior is valid.

The maximum deflection is less than 1/280th of the span and the maximum net deflection is less than 1/400th of the span. It may be concluded that the wall will withstand blasts from as much as 1,000 lb of TNT attenuated 70% and sustain only minor damage. Since the vertical elements will be assisted in carrying blast loads by the horizontal polygonal hoop elements, the actual blast damage would be insignificant and restricted largely to cracking of the periphery of the flat reinforced concrete plate elements forming the wall. Neglecting any blast attenuation, that is, assuming the full effect on the face of the wall of a 1,000-lb TNT bare charge, it was found that the net deflection would be less than 1/100th of the span. Even under this impossible condition the wall would not collapse.

The end reaction of the wall under the conditions of motion described above and taken as a constant force of V_p, or 72 kips per ft, would act outward on the floors above and below the wall for the duration of motion of the ends of the walls. From Eq. 22, this time would be, for the attenuated blast

$$t_1' = \frac{p_o\ t_1}{2\ q_a} = \frac{1700 \times 12 \times 0.649}{2 \times 599} = 11.06 \text{ msec}$$

At the end of this time, the force may be considered to drop abruptly to zero giving the loading function shown in Fig. 11.

It may be visualized that this force acts in the plane of the floor slabs that provide the support against outward radial movement of the side-walls. The floors, subjected to this more or less radially-applied force, tend to tear apart

along vertical radial planes with consequent danger that the sectors of the broken floor may move outward against the containment vessel. The manner in which this bursting tendency is resisted, particularly in the operating floor, is extremely difficult to predict. For simplified and conservative analysis the operating floor may be considered as a hoop which resists bursting through tension in its several components: concrete, reinforcing, steel plates on the underside, and structural supporting members.

To obtain an idea of the amount of floor reinforcement necessary to prevent excessive cracking, a dynamic analysis was made assuming an equivalent hoop of reinforcing steel with a radius of 20 ft. It was assumed that this hoop may be expected to carry only half of the total reaction because of the effective restraint of the other components previously mentioned, the fact that the secondary shield wall itself acts somewhat as a polygonal hoop, and the fact that the load intensity assumed is that for the point closest to the reactor center line, whereas most points are at greater distances. Crossing all radial sections at

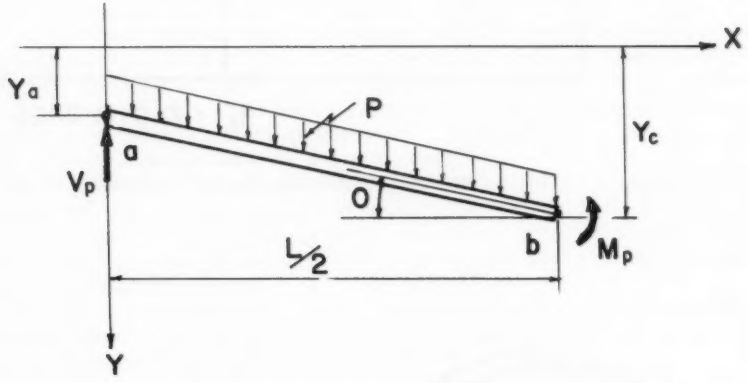


FIG. 10.—COMBINED PLASTIC SLIDING AND BENDING FAILURE.

approximately 20 ft from the building center is the equivalent of at least 20 sq in. of intermediate grade reinforcing steel. This value was used in the analysis.

Assuming the simplified hoop action of Fig. 12, it may be shown that, while the steel remains elastic the radial displacement is

$$x = \frac{V\ r^2}{E\ A} \quad (1 - \cos\ \omega\ t) \quad \ldots\ldots\ldots\ldots (29)$$

in which A is the area of steel in hoop,

$$\omega = \sqrt{\frac{E\ g}{\gamma\ r^2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (30)$$

and γ is the specific weight of steel.

When, at time $t_e \leq t'_1$, the steel becomes plastic, the displacement equation is

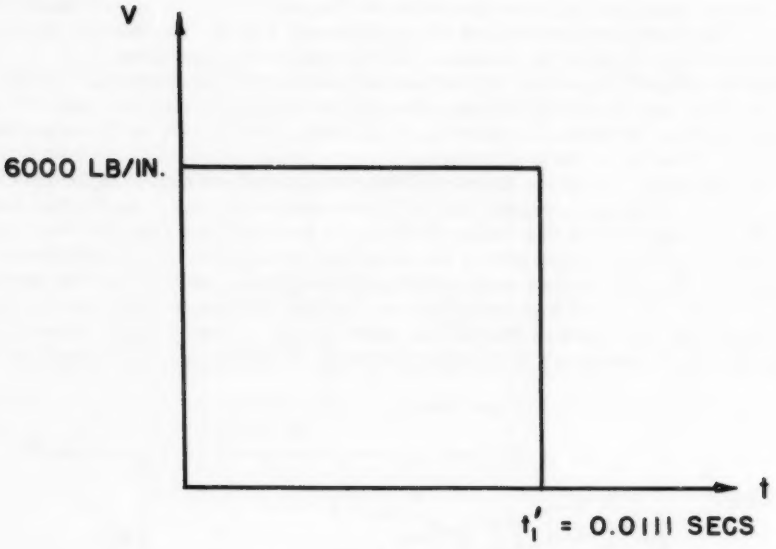


FIG. 11.—FORCE-TIME DIAGRAM FOR OPERATING FLOOR

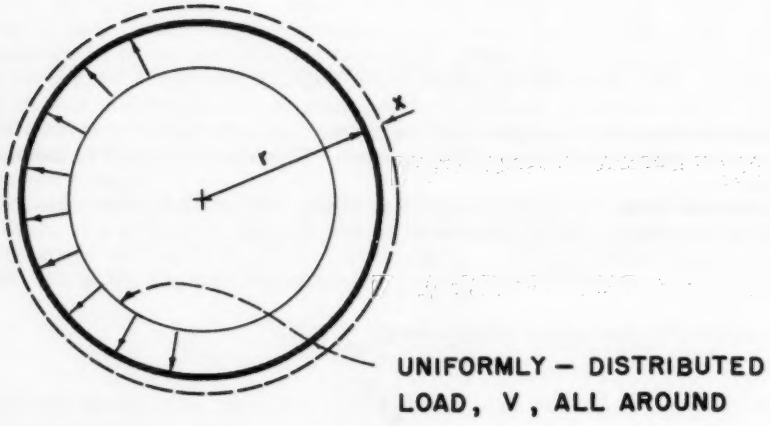


FIG. 12.—HOOP ACTION OF OPERATING FLOOR REINFORCEMENT

$$x = \frac{V\, r^2}{E\, A} \left[\left(t - t_e\right) \omega \sin \omega t_e + 1 - \cos \omega t_e \right]$$

$$+ \frac{1}{2\, m} \left(V - \frac{\sigma_y\, A}{r} \right) \left(t - t_e\right)^2 \quad \ldots\ldots\ldots\ldots (31)$$

in which m refers to mass per running inch of hoop steel and σ_y is the yield point of steel.

Using the conditions that V = 3 k per in., t'_1 = 0.111 sec, σ_y = 40,000 psi, r = 20 ft and A = 20 sq in., it is found that the total computed radial displacement is approximately 1.6 in. This is not excessive since there is an expansion joint between the operating floor and the containment vessel, and a moderate amount of interference is permissible.

The lower floor is not critical. It is more massive than the operating floor and is backed up by concrete fill on the outside of the containment vessel.

Internal Missiles.—As noted previously, the unattenuated air blast from a 1,000-lb TNT explosion, when considered uniformly distributed along a vertical strip of the secondary shield wall, would not cause collapse of the wall. This is the worst conceivable condition to be considered in studying the stability of the wall. The total impulse delivered by the attenuated air blast plus fragments or missiles striking the wall at the same time, cannot be greater than that of the parent blast on the same area.

Several theories on the local piercing action of missiles have been developed but they all require, for quantitative application, the experimental determination of the penetration-resisting properties of the material struck. The best data of this type are those obtained by the armed services in their tests of bomb-resistant structures.

The penetrating effect of a missile can be estimated by the modified Petry formula, used by the Bureau of Yards and Docks, Dept. of the Navy, for designing bomb resistant structures (14,15). This formula is

$$D = k\, A_p \log_{10} \left(1 + \frac{V^2}{215{,}000} \right) \quad \ldots\ldots\ldots\ldots (32)$$

in which D is the depth of penetration in feet, k refers to an experimentally determined material coefficient = 4.76×10^{-3} cu ft per lb for 3,000 psi concrete with 1.4% reinforcement, A_p denotes sectional pressure, that is, missile weight divided by frontal pressure, and V is the striking velocity in feet per second.

The size and maximum velocity of the critical missile resulting from rupture of the reactor vessel in the event of a 1,000-lb TNT equivalent explosion has been estimated by the Naval Ordnance Laboratory on the basis of their classified data as being 2 in. by 6 in. by 10 in. steel block weighing 34 lb and striking the secondary shield wall with a velocity of 903 fps (10). If this missile is assumed to strike with the small face forward, the sectional pressure is 408 psf and the computed penetration is 15.8 in.

The 1 1/2-in. steel plate on the inner surface of the wall provides additional penetration resistance. A reasonable estimate is that structural steel is twelve times as effective as the same thickness of concrete (15), thereby reducing the expected penetration in concrete to about 10 in. Since this is not greater

than one-third the total wall thickness, the wall cannot be breached.[6] Furthermore, the beneficial effect of the primary shield graphite is neglected.

There is additional assurance of the wall's ability to withstand perforation in the results of a bomb shelter test conducted by the Navy Dept. in 1940 (14). A 6-in., 100-lb, pointed projectile was fired at a 31-1/2-in. thick reinforced concrete roof slab at an angle of 21°30', with a striking velocity of 1,023 fps. The slab was of 4,500 psi concrete without steel face plates and had three layers of reinforcing bars in each direction: 5/8-in. round bars at 5 in. spacing one way, 5/8-in. round bars at 10 in. the other. The sectional pressure of the missile was 513 psf. While not identical to the problem under investigation here, in most respects the conditions were remarkably similar and results of the same order of magnitude can be expected. To quote from the Navy report:

> "The 6-inch projectile penetrated the 31 1/2-in. outer roof slab 14 1/2 in., rebounded, and fell in front of the shelter. There were no visible cracks or scabbing on the ceiling inside the shelter. The deflection gage showed a maximum reading of 0.01 in."

The span of the slab was about 16 ft.

In addition to the general correlation between observed and computed depths of penetration, the absence of scabbing on the opposite side of the slab is important since it bears on the question of whether missiles might be formed outside of the secondary shield wall by scabbing of the concrete on the outer side of the secondary wall. The Navy test report indicates that this will not happen even for an unprotected wall. For a wall covered on the back by firmly anchored 1/2-in. steel plates there is no question of scabbing or of rupture of the outer plate. The Navy report states further that anti-scabbing plates were effective in increasing perforation resistance in their tests. It may be concluded that breaching or scabbing of the secondary shield wall or the thicker operating floor above is not a problem. Furthermore, the few random missiles which might approach the 10-in. penetration depth at intervals on the wall would not impair the overall dynamic strength of the wall because of their localized effect.

CONTAINMENT VESSEL

Design.—Design and construction of the containment vessel follow the best current practices:

1. The hemiellipsoidal bottom is completely encased in a massive concrete mat bearing directly on rock. There is no possibility of settlement which would cause unforeseen stresses in the vessel.
2. There is a transition section between the full encasement and the free section of the vessel which will prevent the development of large bending stresses possible at a sharp discontinuity in supports.
3. The steel fabricators who designed the vessel have accounted for all reasonable operating loads (dead, crane, wind, snow and temperature). They have also checked the capacity of the structure to resist an earthquake of very heavy magnitude for the Michigan area. They have accounted for bending at constraints as well as primary membrane stresses.

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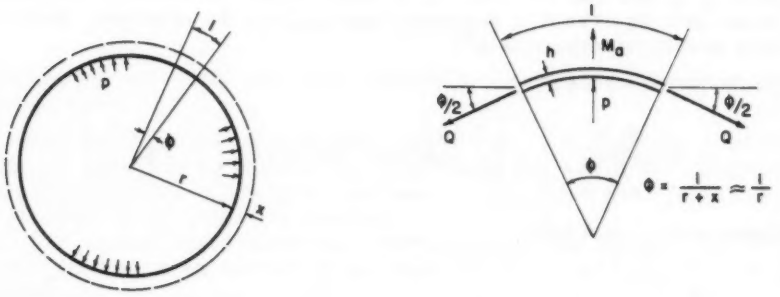
[6] Bibliography reference (15) states that the Petry formula is directly applicable only when the computed penetration is less than 1/3 the thickness of the wall. For computed penetrations greater than 1/2 the wall thickness, breaching may occur.

4. The designers have accounted for the maximum internal pressure of 32 psig originally specified as that which might be expected in the event of a sodium fire. They have also provided for a specified external pressure of 2 psig with vacuum relief for pressures above this, another condition which might result from a sodium fire. It would appear that the use of allowable stresses specified by the ASME code for vessels under pressure during normal operation is a conservative practice for containment vessels which are under pressure only in the event of an unlikely accident.

5. The use of normalized killed steel with a minimum Charpy impact resistance of 15 ft-lb at -50° F, full penetration butt welding and stress relieving wherever possible, and the avoidance of serious stress raisers, afford the best means of protection known at present against brittle fracture, and provide excellent resistance against normal operating loads or emergency condition dynamic loads. Several exhaustive surveys of brittle failure in actual structures have brought to light no such failures in the type of steel used for the containment vessel (16,17).



(a) SECTION OF CYLINDER (b) UNIT ELEMENT

FIG. 13.—ACTION OF CONTAINMENT VESSEL

Blast Containment.—Little or no air blast effect is expected on the outer shell, but the shell was analyzed for the blast loading from a 1,000-lb TNT explosion, attenuated 70%, to obtain an extreme upper limit of damage.

It was assumed that any blast wave could exert maximum pressure on the shell in the area below the operating floor. In this region the shell is a simple cylinder. It may be treated as an elemental hoop dynamically loaded by the internal shock wave. The vertical stresses in the shell wall have been neglected in the dynamic containment analysis. Consider a unit length of shell (Fig. 13). Summing forces in the radial direction

$$p - 2\,Q\,\frac{\phi}{2} = M\,a \quad \ldots\ldots\ldots\ldots\ldots\ldots (33)$$

$$p - \frac{Q}{r} = M\,a \quad \ldots\ldots\ldots\ldots\ldots\ldots (34)$$

Assuming a triangular pressure pulse,

$$p = p_o \left(1 - t/t_1\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (35)$$

and

$$Q = \sigma h$$
$$M = h(\gamma/g)$$
$$a = d^2x/dt^2$$

in which σ is the unit stress in the ring, σ refers to the $E\epsilon$ for elastic range, σ is the σ_y (yield point) for plastic range, and γ = specific weight of steel. Then, by substitution in the force equation,

$$\frac{d^2x}{dt^2} + \frac{g\,\sigma}{\gamma\, r} = \frac{p_o\, g}{\gamma\, h}\left(1 - \frac{t}{t_1}\right) \quad \ldots\ldots\ldots\ldots (36)$$

Integration of this equation and application of the boundary conditions yields two sets of equations for the radial displacements, x, as a function of time: one if t_1 is less than the time, t_e, at which elastic action stops and yielding begins, and the other if t_1 is greater than t_e. For the first case, which was found to hold, the equations are:

Elastic action. $-0 \leq t \leq t_1$

$$x = \frac{p_o\, r^2}{E\, h}\left[\frac{1}{\omega\, t_1}\ \sin\,\omega\, t - \cos\,\omega\, t + \left(1 - \frac{t}{t_1}\right)\right] \quad \ldots\ldots (37)$$

Elastic action. $-t_1 \leq t \leq t_e$

$$x = \frac{p_o\, r^2}{E\, h}\left[\frac{1}{\omega\, t_1}\left(1 - \cos\,\omega\, t_1\right)\sin\,\omega\, t - \left(\frac{\omega\, t_1 - \sin\,\omega\, t_1}{\omega\, t_1}\right)\cos\,\omega\, t\right] \ldots (38)$$

Plastic action. $-t_e \leq t$

$$x = \frac{p_o\, r^2}{E\, h}\left\{\left[\frac{\left(t - t_e\right)}{t_1}\ \cos\,\omega\, t_e + \frac{\sin\,\omega\, t_e}{\omega\, t_1}\right]\left[1 - \cos\,\omega\, t_1\right]\right.$$
$$\left. + \left[\frac{\left(t - t_e\right)}{t_1}\ \sin\,\omega\, t_e - \frac{\cos\,\omega\, t_e}{\omega\, t_1}\right]\left[\omega\, t_1 - \sin\,\omega\, t_1\right]\right\}$$
$$- \frac{\sigma_y\, g}{2\,\gamma\, r}\ \left(t - t_e\right)^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots (39)$$

in which

$$\omega = \sqrt{\frac{E\, g}{\gamma\, r^2}}$$

For a 300-lb TNT blast (6), for a radius of 36 ft:

Face on overpressure, p_o = 84 psi
Face on specific impulse = 0.182 lb. sec/in.^{2}
Assuming linear pressure decay, t_1 = 4.35 ms.
σ_y = 32,000 psi
E = 30 x 10^6 psi
h = 1.03 in. = shell thickness

It was found from the preceding equations that the radial expansion would stop while the shell is still elastic at a time of 4.78 msec. The maximum expansion would be 0.459 in., and the computed maximum stress 31,800 psi. Thus the containment vessel would not yield but would vibrate radially within the stated range until damped out. For this condition the maximum rate of strain may be shown to be 0.34 in. per in. sec. This is not an excessive strain rate of the type that might produce fracture. At this rate an increase in the yield point of at least 15% might be expected (18), further guaranteeing elastic behavior.

ADDITIONAL EFFECTS

In addition to the blast effects summarized previously, the following problems were studied:

1. The motion of the rotating plug, control rods, and materials handling equipment caused by air blast or static pressure rise within the secondary shield wall enclosure following an explosion. It was found that none of these objects could be converted into missiles.
2. The transmission of shock waves through the sodium piping and the tendency to fracture it and convert it into missiles. This was found not to be a problem.
3. The effect of air blasts on equipment outside the secondary shield enclosure, that is, heat exchangers, sodium pumps, piping, the overhead crane, etc. It was found that these would not be detached from their supports.
4. The effect of a sodium fire in heating exposed steel surfaces to the danger point. Recommendations were made for moderate fireproofing of the steel in certain areas.
5. The possibility of an external missile such as an aircraft or portion of one striking the containment vessel. Probability studies showed that this is a negligible factor. Even if one did hit, it would have to be extraordinarily heavy or have a very high velocity to penetrate below the operating floor where it might damage the reactor.

CONCLUSIONS

The major conclusions regarding blast effects can be summarized. In the event of a nuclear excursion equivalent to the detonation of 1,000 lb of TNT at the reactor core the following is expected to happen:

1. The blast wave will deform the secondary shield wall permanently. It may bend and yield at about mid-height and will crack along the edges. The steel face plates and the embedded structural and reinforcing steel will keep the permanent deformations within tolerable limits.

2. The operating floor may crack because of the dynamic loads from the secondary shield wall. However, with the reinforcement provided, the outward movement of the operating floor will not endanger the containment vessel.

3. The reactor, reactor vessel, sodium piping, borated graphite, and primary shield tank in the region of the reactor core may fracture and form missiles. However, these missiles will be contained within the secondary shield wall and will not endanger the containment vessel.

4. The shock wave will not cause the rotating plug and control rods to form dangerous missiles.

5. Missiles will not be formed outside of or above the secondary shield wall region.

6. There will be very little if any shock effect on the outer containment vessel. It is capable of containing any possible residual shock without deforming plastically.

7. The post-blast static pressures and temperatures in the outer vessel cannot be as serious as those for which the vessel was designed.

ACKNOWLEDGMENTS

The study summarized in this paper was made by the authors as consultants to Power Reactor Development Company, builders and operators of the Enrico Fermi Plant. Design of the reactor and initial testing prior to nuclear operation are under the jurisdiction of Atomic Power Development Associates, Inc., the associated design and development organization. Output will be delivered to an adjacent 150 MW turbine - generator plant being built by the Detroit Edison Company. The structural design was prepared by Commonwealth Associates, Inc., with the exception of the outer steel containment vessel, designed and erected by the Chicago Bridge and Iron Company. Additional studies were made by the Cornell Aeronautical Laboratory.

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Proceedings of the American Society of Civil Engineers

DESIGN OF SELF SUPPORTED STEEL TRANSMISSION TOWERS

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SYNOPSIS

Present (1959) tower design practices are reviewed with comments on existing codes. The economy of using various materials such as A-7 steel, high strength steel, and aluminum is examined. The design of a transmission line is outlined, illustrating how numerous repetitions of the same tower type offered an opportunity to effect considerable savings through careful analysis.

INTRODUCTION

The cost of constructing high voltage transmission lines is a matter of great concern to the electrical industry. This has become a major item in the general problem of reducing all costs contributing to the expense of generating and distributing electrical energy. This problem receives considerable emphasis because of the present (1959) attempts to better serve people over wide areas by maintaining interconnections between utilities to meet local concentrated load demands and because of increased emphasis on the desirability of locating generating stations near sources of fuel supplies to combat rising freight rates.

This paper examines the present practices used in the design of these towers, with particular emphasis upon the points in which these practices differ from conventional practices used in the design of buildings or other structures. In

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reviewing these practices, it should be borne in mind, that any slight saving of weight on one tower offers a major saving in cost in cases where this type of tower is used many times on a transmission line of considerable length.

Following examination of the general principles of tower design, this paper reports, in detail, on the design of a self-supported transmission tower based on a criterion for obtaining the utmost economy consistent with adequate safety. Included are such topics as the basis for selection of tower configuration, selection of loading conditions, stress analysis, selection of members and a report of an actual destruction test on a full size prototype of the basic tower.

PURPOSE OF TOWER

A transmission line is used to transmit power from one location to another, often hundreds of miles away. The line may consist of either one, two, or three circuits with three phase wires comprising each circuit. There may be also static wires, sometimes called shield or ground wires, for the purpose of shielding or grounding the charge if the line is struck by lightning. Supporting these wires, are towers placed at intervals in the line with a typical arrangement for a two circuit tower shown in Fig. 1.

Towers may be of either 1) tangent types, 2) angle types, or 3) dead end or strain types. Tangent towers, used in the straight sections of the line, are designed to support the wires when subject to wind and ice loads and frequently are designed for a small angle in the line. They may or may not be designed for the unbalanced pull due to a broken wire on one side of the tower.

Angle towers, placed at points where the line changes direction, are designed to take the same loads carried by the tangent tower with the addition of the transverse pulls due to the angle in the line. In order to utilize the structures most efficiently they are usually placed so that the axis of the cross arm bisects the angle formed by the conductors as shown in Fig. 2.

Dead end, strain, or anchor towers are designed to take dead end pulls from all the wires on one side, together with wind and vertical loads.

HISTORY OF TOWER DEVELOPMENT

Steel towers were first used for overhead high tension transmission in a line 100 miles long in Mexico, in 1903, where windmill towers were modified to form supports for the wires. Steel towers are now used on most important lines. Early steel structures were often of inefficient design and supported short spans. Today they are highly efficient structures and have kept pace with electrical progress. There is a line spanning 3,800 ft over the Sacramento River in the United States, with towers 437 ft high. The world's longest crossing is over the Messina Straits, between Sicily and the Italian mainland. The wires carried by these towers span 12,000 ft, and the towers are more than 700 ft high. These are unusual and not typical examples. Normal lines have towers from 100 ft to 150 ft high and support wires spanning 700 ft to 1,400 ft. One of the longest lines in the United States extends 477 miles from a steam power plant in the West Virginia coal region through Ohio and Indiana to Chicago, Illinois.

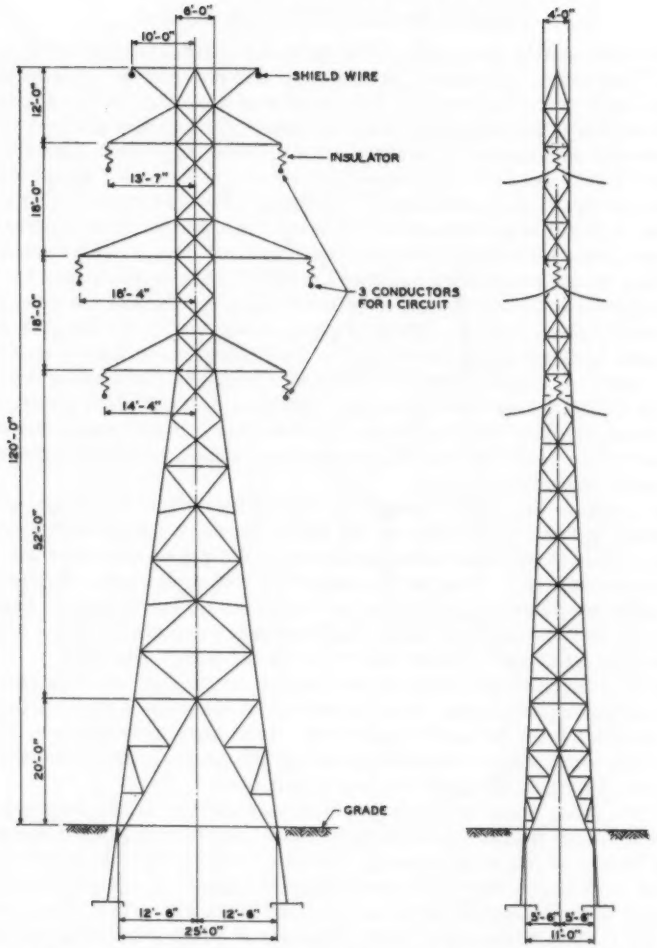

FIG. 1.—ELEVATION OF 2 CIRCUIT 230 KV TRANSMISSION TOWER

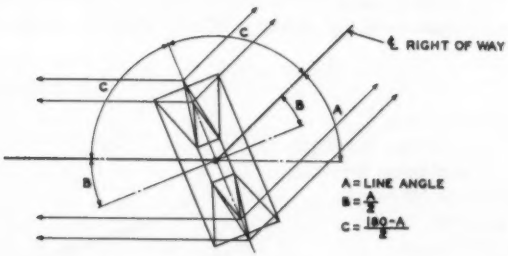

FIG. 2.—PLAN OF TOWER

CODES AND CRITERIA FOR DESIGN

Towers are usually designed according to the National Electric Safety Code[4] (NESC). The code, in general, is advisory and not mandatory except where state laws have so recognized it. This code was a product of the gradual evolution from early transmission lines on wood poles to the present day steel structures and was generally written by electrical engineers concerned with that type of work. As such, it presents an excellent guide for electrical items and covers in detail such subjects as conductor size and material, selection of insulators, vertical separation between insulators, and various clearances, but it presents little real information for the structural design of the tower. Illustrating this, it can be pointed out that the NESC uses seven pages to discuss climbing space requirements but only three pages to discuss the entire structural design of steel towers. This can only mean that many important design factors must be determined according to the discretion of the designer.

Tower Shape or Configuration.—The tower shape is dictated by the number of circuits and the clearances required for these, and the clearances required to the ground or other obstructions. Within these limits some variation in tower shapes can be developed and the designer should investigate these to select the most economical one.

Design Loads.—The NESC divides the United States into three zones of loading as shown in Fig. 3. Towers in the heavy zone are designed for 1/2 in. of radial ice on the wires and a wind pressure of 6.5 psf on the tower and 4.0 psf on ice-covered wires. This is equivalent to a 40 mph wind. Towers in the medium zone are designed for 1/4 in. of radial ice on the wires and same wind pressure as used in the heavy zone. Light zone towers are designed for no ice with a 14.5 psf wind on the tower and 9 psf on the wires (60 mph).

Known local conditions, such as hurricanes or heavier ice, can change any of these loading requirements. It is known that at least one Florida utility company designs its lines for a 145 mph wind. It is also interesting to note that under certain conditions a tower designed for the heavy loading conditions will not completely satisfy the light loading conditions.

One of the many ways in which tower design differs from ordinary bridge and building design is that tower design utilizes overload factors to incorporate factors of safety in place of reducing the allowable stress. In tower work the anticipated load is arbitrarily increased and the tower is designed to stresses approaching the yield point, while in building work the actual loads are used and the structure is designed to lower stresses at some percentage of the yield point.

The overload factors by which towers are normally designed are specified in the NESC. The actual tower loads must be multiplied by these factors to obtain the fictitious load used for design. These factors are as follows:

	Overload Capacity Factor
Vertical Strength	1.27
Wind Transverse Strength	2.54
Angle Transverse Strength	1.65
Longitudinal Strength:	
In general	1.00
At crossings	1.10
At dead ends	1.65

[4] National Electric Safety Code, 5th Edition, U. S. Department of Commerce, Part 2, 1949.

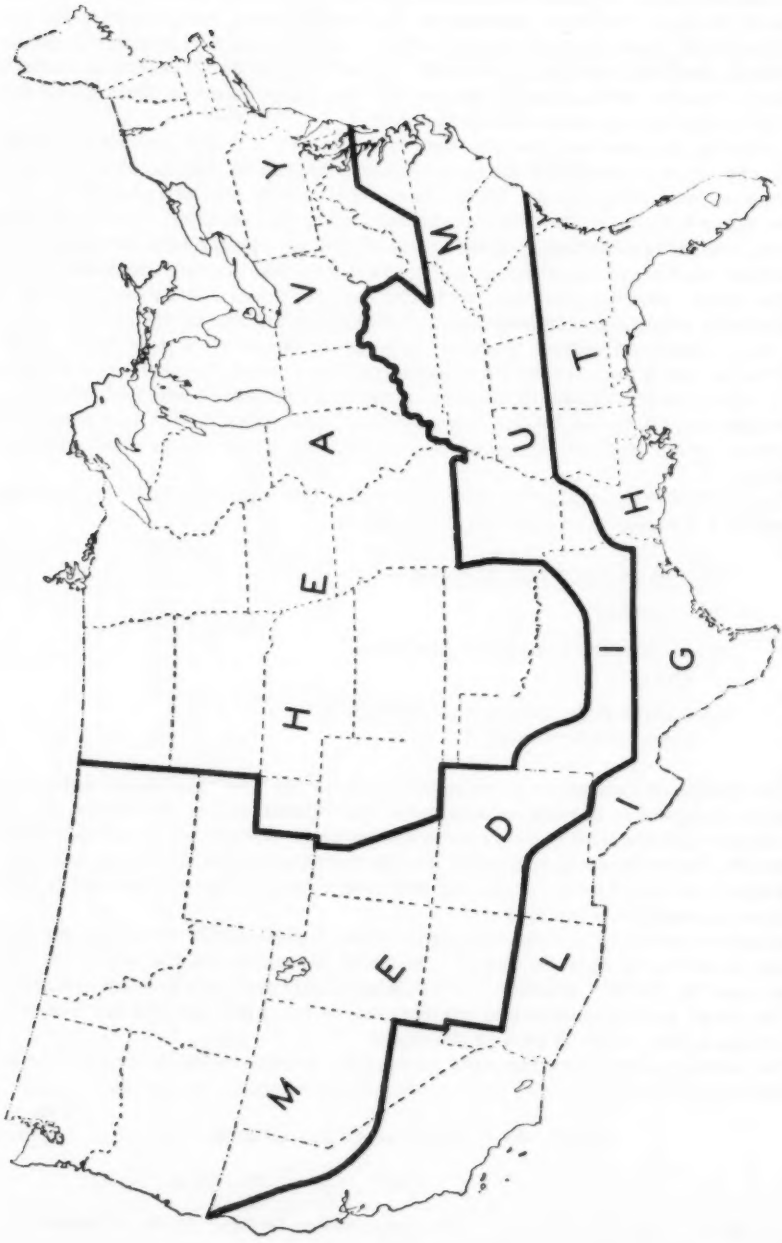

FIG. 3.—TERRITORIAL DIVISION OF U. S. FOR LOADING OF OVERHEAD LINES

The reason for this overload factor approach to design is not discussed in the code but the usual explanation for load factors is that the safety factor of each item of strength, vertical, transverse, and longitudinal, can be controlled according to its importance in the structure. Overload factor design has disadvantages, however, in that, it promotes the belief that the structure is working at high stresses when actually the results are little different from those that would be obtained by conventional methods.[5]

Utilizing the overload factors described previously and the basic loading data, the tower is designed for various combinations of the different types of loading as prescribed by the NESC. The combinations that are prescribed, include some for which the reasons are difficult to understand. One provision is that the tower standing as a structure alone, be designed for six times the specified wind pressure, although the wires are subject to the same wind forces as the tower. Another provision provides that the tower can be designed for a condition in which only the longitudinal pulls along the axis of the transmission line are considered without transverse loads or the vertical dead loads of the structures and wires. These provisions, perhaps, were not serious in former days, when towers were designed for generous overload conditions with considerable margin in the loading assumptions. But today (1960) they should be reviewed in the light of trends towards reducing tower weights as much as possible.

Design Criteria.—As given in NESC code, the following are the limiting values of l/r ratios for compression members:

Kind of Compression Member	l/r
Leg members	150
Other members carrying computed stress	20
Secondary members without computed stress (redundants)	250

The design of compression members is less precise than is the design of tension members. In tension members, the effects of eccentricities due to fabrication defects tend to decrease with increase in load, but in compression members, the reverse is true. The design of compression members has been the subject of considerable study and test over a long period of time and it still remains an empirical method.

No allowable ultimate compression formula is given in the NESC and the designer is left to his own discretion. Some of the numerous formulae that are being used by utility companies, consulting engineers, and steel fabricators will be listed and the number and variety clearly bring out the need for a standard formula that could be part of the NESC.

The compression formulae used by various designers for A-7 steel are as follows:

$$16200 - 60\,\frac{l}{r}\ (12600\ \text{max})\ \text{For Bracing} \quad \ldots\ldots\ldots\ (1)$$

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[5] "Overload Factors Can Cause Ultra-Conservative Design," by R. N. Bergstrom, Proceedings, Vol. 85, No. ST 2, February, 1959.

$$17000 - .485\left(\frac{l}{r}\right)^2 \quad \text{For } \frac{l}{r} < 120 \quad \ldots\ldots\ldots\ldots\ldots\ldots \text{(2a)}$$

$$\frac{18000}{1 + \dfrac{l^2}{18000\, r^2}} \qquad \left(\frac{320 - l/r}{200}\right) \quad \text{For } \frac{l}{r} > 120 \quad \ldots\ldots \text{(2b)}$$

$$23600 - 80\,\frac{l}{r} \text{ (19000 max.) Susp. Tower Legs} \quad \ldots\ldots\ldots \text{(3a)}$$

$$36100 - 122\,\frac{l}{r} \text{ (30000 max.) Tower Legs, Broken Wire} \quad \ldots \text{(3b)}$$

$$32000 - 130\,\frac{l}{r} \text{ (22000 max.) Tangent Tower Legs} \quad \ldots\ldots \text{(4a)}$$

$$27000 - 115\,\frac{l}{r} \text{ (18500 max.) D. E. Tower Legs} \quad \ldots\ldots\ldots \text{(4b)}$$

$$33000 - 130\,\frac{l}{r}\left(\frac{l}{r} < 150\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots \text{(5a)}$$

$$24000 - 70\,\frac{l}{r}\left(\frac{l}{r} < 150\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots \text{(5b)}$$

$$3000 - 100\,\frac{l}{r} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(6)}$$

$$38000 - 165\,\frac{l}{r} \text{ (30000 max.) } \frac{l}{r} < 150 \quad \ldots\ldots\ldots\ldots \text{(7a)}$$

$$28000 - 100\,\frac{l}{r}\left(\frac{l}{r} > 150\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots \text{(7b)}$$

$$\frac{22000}{1 + \dfrac{l^2}{1500\, r^2}} \quad \text{(17600 max.) For Line Towers} \quad \ldots\ldots\ldots \text{(8a)}$$

$$\frac{20000}{1 + \dfrac{l^2}{15000\, r^2}} \quad \text{(16000 max.) For Crossing Towers} \quad \ldots\ldots \text{(8b)}$$

while the variety of compression formulae used for high strength steel are as follows:

$$43000 - 170\,\frac{l}{r}\left(\frac{l}{r} < 150\right) \ldots \quad \ldots\ldots\ldots\ldots\ldots\ldots \text{(9a)}$$

$$39000 - 145\,\frac{l}{r}\left(\frac{l}{r} > 150\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(9b)}$$

$$33150 - 150 \frac{l}{r} \quad (24000 \text{ max.}) \ldots\ldots\ldots\ldots\ldots\ldots \quad (10)$$

$$34000 - 140 \frac{l}{r} \quad (25000 \text{ max.}) \left(\frac{l}{r} < 150\right) \ldots\ldots\ldots\ldots \quad (11)$$

$$50950 - 206\ l/r \quad (37500 \text{ max.}) \ldots\ldots\ldots\ldots\ldots\ldots \quad (12)$$

$$40000 - 1.3 \left(\frac{l}{r}\right)^2 \quad \text{For } (44 - 113) \frac{l}{r} \ldots\ldots\ldots\ldots\ldots\ldots (13a)$$

$$\frac{40000}{.5 + \frac{(l/r)^2}{10500}} \quad \text{for } (114 - 200) \frac{l}{r} \ldots\ldots\ldots\ldots\ldots\ldots (13b)$$

The impressive and varied list of formulae cited is an excellent illustration of the freedom of choice and the resulting confusion existing in the tower design field. A graphical representation of the variation in allowable stresses resulting from this confusion is shown in Figs. 4 and 5. The fact that such variations exist with experienced designers illustrates the dangers that could develop if an inexperienced designer is permitted complete freedom of choice.

No limiting value for l/r in tension is given in the NESC, whereas under the American Institute of Steel Construction (AISC) code,[6] it is limited to 240 for main members and to 300 for bracing and other secondary members in tension. As used in the tower industry, the range of l/r for tension members is from 375 to 600 depending on the designer. Here again, there is need to incorporate restrictive l/r ratio for tension members in the code. Frequently, tension members, acting as part of an X bracing system as shown in Fig. 6, are called upon to act as redundant bracing for the compression leg of a tower and cannot act as part of a compression bracing system if they are too limber.

These points illustrate the fact that the present provisions of the NESC are not adequate to cover the structural design of the tower. The code is correct in avoiding discussion of methods of design analysis as these are quite properly left to the discretion of the designer, but the code is not meeting its proper function in prescribing criteria normally considered in the province of such codes.

Deviations from Standard Practices in Structural Design.—Transmission towers are almost invariably constructed of angle sections. Joints usually are lapped spliced, with no recognition being made of eccentric stress transfer because of the lap splice. However, splices are kept as close as possible to a panel point in order to minimize the effect of eccentricity. Bracing members are often connected together by one leg only and usually no reduction is made to allow for the outstanding leg of the angle being unconnected, although some designers do consider only one half of the outstanding leg in their design.

These are the major design deviations from standard practice and while actual test results on a full scale tower often show that these deviations are not critical, it is important that connection details be properly made and eccentricity kept to a minimum.

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6 Steel Construction, 5th Edition, Amer. Inst. of Steel Cons., 1955.

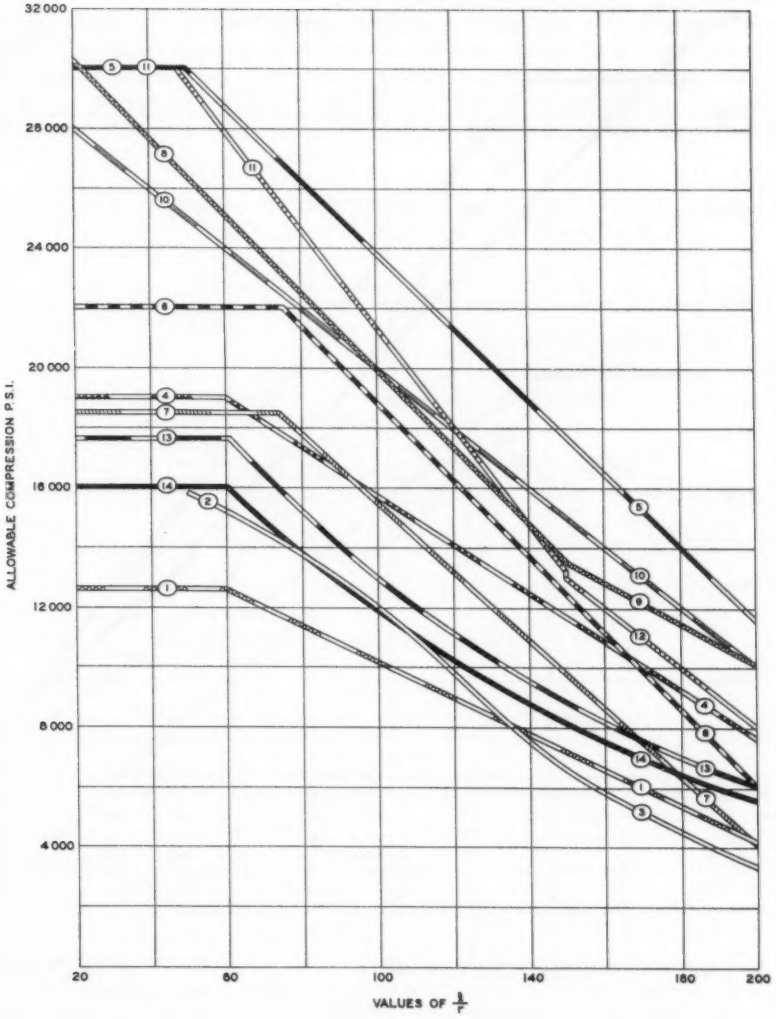


FIG. 4.—WIDE RANGE OF ALLOWABLE COMPRESSION VALUES FOR A7 STEEL

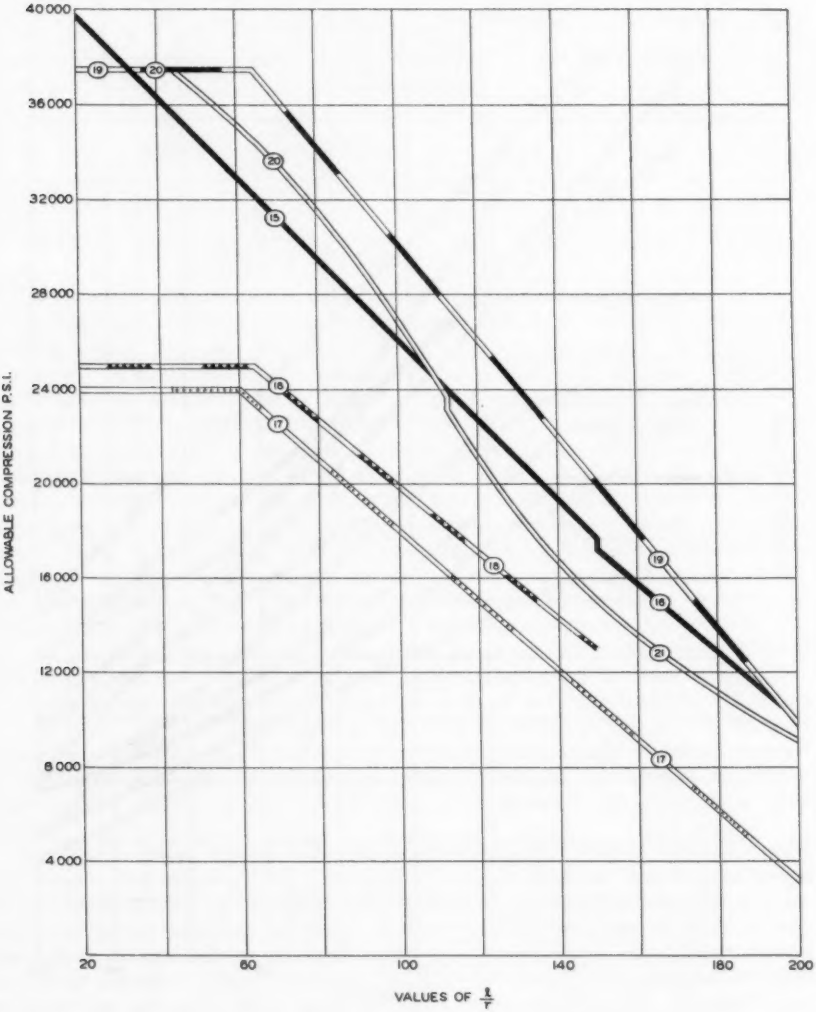


FIG. 5.—WIDE RANGE OF ALLOWABLE COMPRESSION VALUES FOR HIGH-STRENGTH STEEL

MATERIALS

Materials normally considered for transmission tower structures are A-7 structural steel, A-242 high strength steel and aluminum. Under present day (1959) prices aluminum is usually ruled out because, although it weighs almost a third the weight of steel, it is much weaker in compression, particularly in the high l/r ranges and its basic cost at this time is estimated to be approximately 4.2 times the cost of steel per pound of material. Table 1 illustrates this.

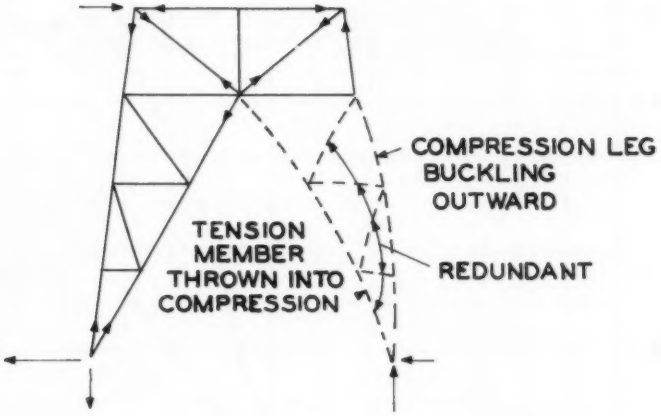


FIG. 6.—DEFORMATION OF COMPRESSION LEG

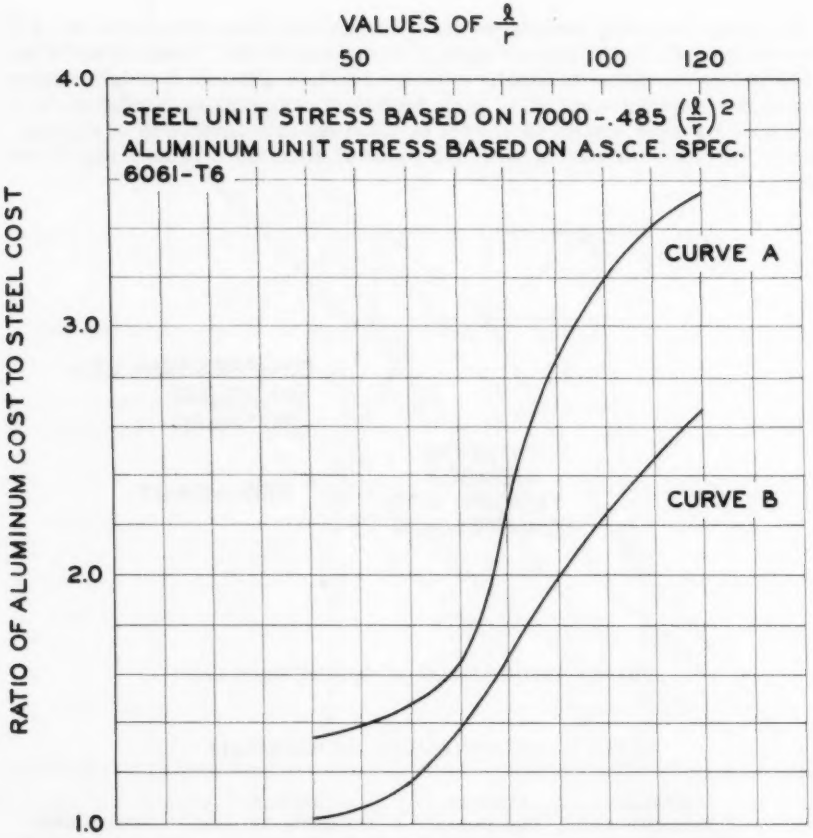
TABLE 1.—RELATIVE COSTS OF MATERIALS

l/r	Allowable Compressive Stress Aluminum	Allowable Compressive Stress Steel	Ratio of Allowable Stresses Steel/Aluminum	Ratio of Cost Steel/Aluminum
50	11,000 psi	15,788 psi	1.43	.71
120	2,800 psi	10,016 psi	3.57	.28

Fig. 7 shows the cost of aluminum shapes in compression as compared to A-7 steel, based on the allowable $f_c = 17{,}000 - .485\ (l/r)^2$ for steel and a progress report of the American Society of Civil Engineers,[7] for aluminum assuming a 4.2-to-1 ratio for the cost of aluminum to steel on a weight basis.

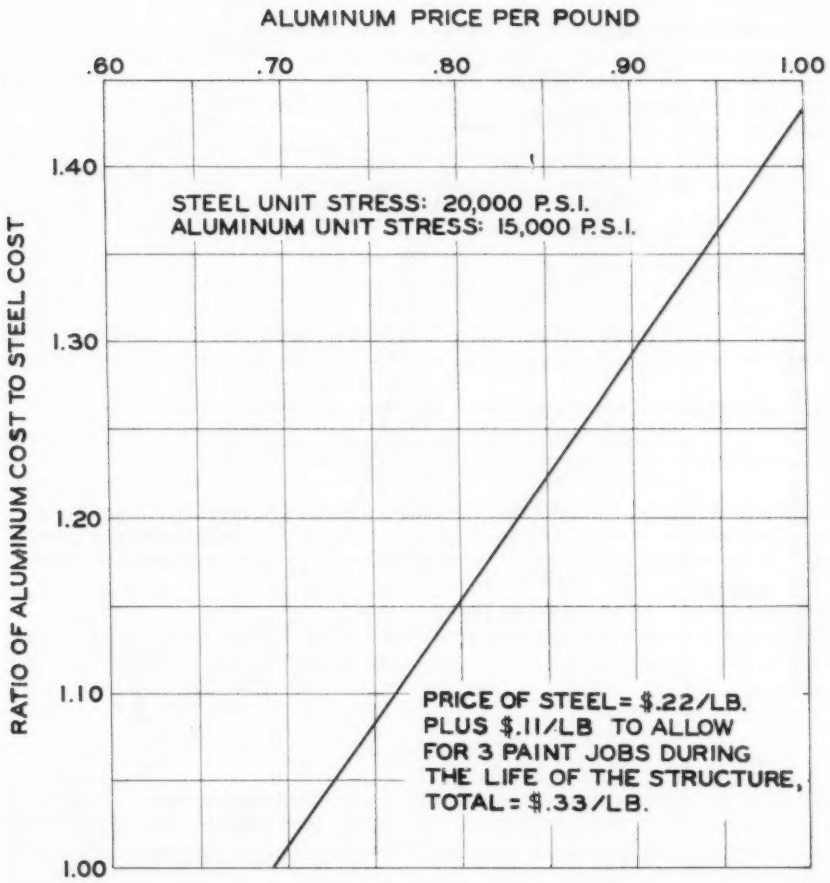
In tension members, the cost ratio is not as great as in compression members. The curve in Fig. 8 illustrates this point. Cost ratio of aluminum to steel is plotted against various aluminum prices. The curve shows that for tension members, aluminum is as economical as steel, only if a price of $.69 per pound for fabricated aluminum is secured as compared to an assumed price of $.22 per pound for fabricated steel. High strength steel was found to be very

[7] "Specifications for Structures of Aluminum Alloy," Proc. Paper Nos. 970 and 971, ASCE.



1. CURVE A: BASED ON COST OF ALUMINUM = $.925/LB. FABRICATED.
   CURVE B: BASED ON COST OF ALUMINUM = $.69/LB. FABRICATED.
2. FOR BOTH CURVES: COST OF STEEL = $.22/LB. PLUS $.11/LB. TO ALLOW FOR 3 PAINT JOBS DURING THE LIFE OF THE STRUCTURE, TOTAL = $.33/LB.
3. WEIGHT OF ALUMINUM = $\frac{\text{WEIGHT OF STEEL}}{2.82}$
4. EXAMPLE FOR CURVE A: $\frac{\ell}{r}$ = 100, ALUMINUM WILL COST 3.2 TIMES COST OF STEEL.
5. EXAMPLE FOR CURVE B: $\frac{\ell}{r}$ = 100, ALUMINUM WILL COST 2.25 TIMES COST OF STEEL.

FIG. 7.—COST OF ALUMINUM SHAPES IN COMPRESSION COMPARED TO A-7 STEEL



EXAMPLE:
AT $ .69/LB. ALUMINUM WILL COST SAME AS STEEL.

AT .90/LB. ALUMINUM WILL COST 1.28 TIMES COST OF STEEL.

FIG. 8.—COST OF ALUMINUM SHAPES IN TENSION COMPARED TO A-7 STEEL

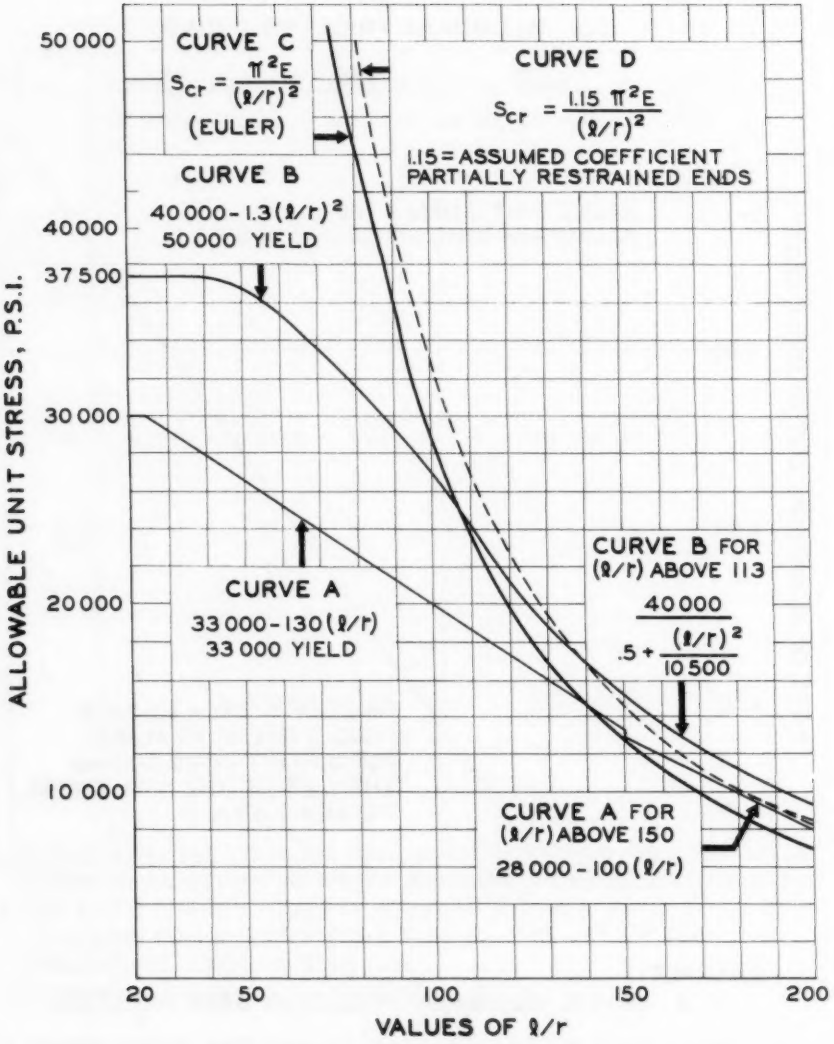


FIG. 9.—COMPRESSION CURVES BASED ON EULER'S, A-7 AND HIGH-STRENGTH STEEL FORMULAS

economical in compression members of low slenderness ratio, making it ideally suitable for tower legs.

Fig. 9 shows graphs plotted for the "ultimate" compression formulae used subsequently in this paper for A-7 steel and high strength steel (50,000 psi yield). The "C" curve shows the Euler curve for pinned ends and the "D" curve shows the Euler curve for slightly restrained ends. The chart shows that for high strength steel members with $l/r > 140$, Euler's curve for slightly restrained members governs, and curve "B" can no longer be used. Hence, the economy of high strength steel becomes less and less as l/r values continue to rise. For values above $l/r = 140$, high strength steel is no better in compression than any other steel having a modulus of elasticity of 29,000,000, regardless of the value of the yield point.

Lacing members generally fall in the $l/r = 140$ range and, therefore, should be A-7 steel. Since all stressed members are generally connected by a minimum of 2 bolts, Euler curve showing slightly restrained ends can be used as a limit with a formula such as the $28,000 - 100\ l/r$ compression formula used up to that point. This type of assumption was verified and justified by tower tests that plainly showed the members deforming into an "S" curve with restrained ends. This can be seen in Fig. 10.

In members acting in tension only, high strength steel can be economical only in highly stressed members. In low stress members, the size of a tension member is generally controlled by the bolt connection requirement and l/r considerations.

Redundant members are chosen primarily for their l/r ratios, and, therefore, there is no point in using high strength steel for these. Because of factors such as these, it is not economical to design a tower entirely of high strength steel.

Fig. 11 shows the comparison between high strength and A-7 steel required for compression members by plotting the relationship between compression slenderness ratio, l/r, and K, where K is defined as the ratio of area of high strength steel required to the area of A-7 steel required.

TOWER DESIGN

Design of 230 kv Tangent Tower.—In a recent design of a 230 kv transmission line, approximately 100 miles long, careful attention was given to all points that could contribute to reducing the weight of the tower to the lightest weight consistent with safety. A check on current prices revealed that high strength steel having a yield of 50,000 psi and an ultimate tension of 72,000 psi would cost only $25.00 per ton more than A-7 steel. This slight difference made the use of high strength steel worthwhile in the areas described. Since the line will contain approximately 500 tangent towers it can be seen that each pound of steel saved per tower saves approximately $100 over the length of the line. It was evident, therefore, that a major savings could be developed by a careful thorough design with emphasis on economy. It was agreed that the tower would be designed for two conditions:

1. Stringing condition during wire stringing.
2. Heavy loading condition during operation.

For the stringing condition, the vertical, wind, and transverse pull due to angle are less than for the operating condition. However, a longitudinal pull of 5,000

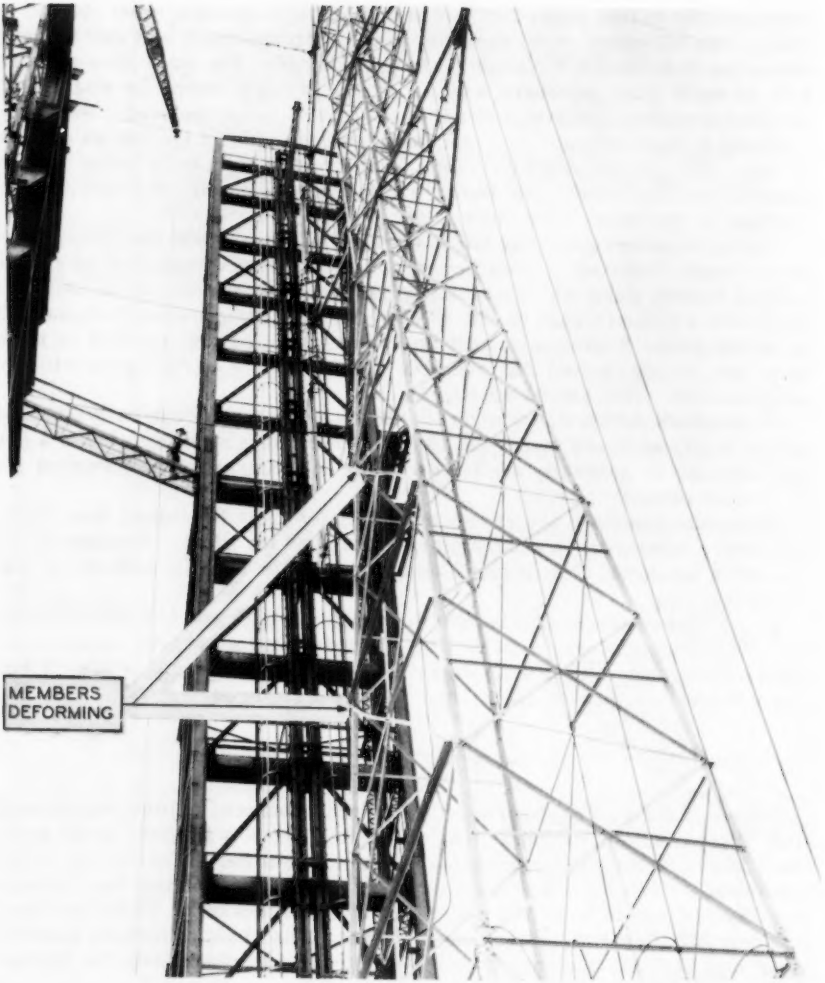


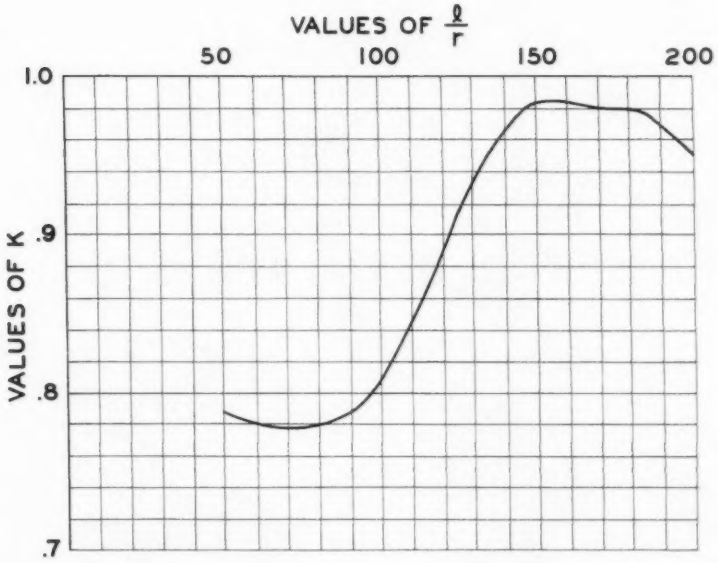
FIG. 10.—MEMBERS DEFORMING INTO AN S CURVE DURING TEST

lbs was applied at any wire attachment to provide for the pull on the tower in case the cable is hung up during the stringing operations. The overload factor applied to the stringing condition loads is only 1.1. In other words, any member whose design is governed by the stringing condition has a factor of safety in it of 10%. This was felt to be justified.

Although the selection of the 5,000 lb pull was an arbitrary one, it was thought to be conservative with very little likelihood of occurring. Actual construction of the line is complete and the assumption was proven to be conservative.

The loads present during the operating condition are due to:

1. 1/2 in. ice covered wires times 1.27.
2. Wind on wires at 4 psf times 2.54.
3. Wind on tower at 6.5 psf times 2.54.
4. Transverse pull due to wire tension for 2° line angle times 1.65.
5. Weight of tower times 1.27.



1. K AREA OF $\frac{\text{A-242}}{\text{A-7}}$ MEMBER
2. K HAS BEEN INCREASED 10 % TO MAKE UP FOR $25 / TON PRICE DIFFERENTIAL FOR A-242 STEEL

FIG. 11.—ECONOMY OF USING 50,000 P.S.I. YIELD (A-242 R) AS COMPARED TO A-7 STEEL FOR COMPRESSION STEEL

Fig. 12 shows the stringing condition loads with the longitudinal stringing pull applied at the ground wire cross arm. Fig. 13 shows the operating condition loads. Figs. 14, 15, and 16 show the stringing condition with the stringing pull applied successively at the bottom, middle, and top conductor arms.

Examination of these loading conditions reveals that the tower need have no longitudinal strength under heavy loading conditions. Its longitudinal strength

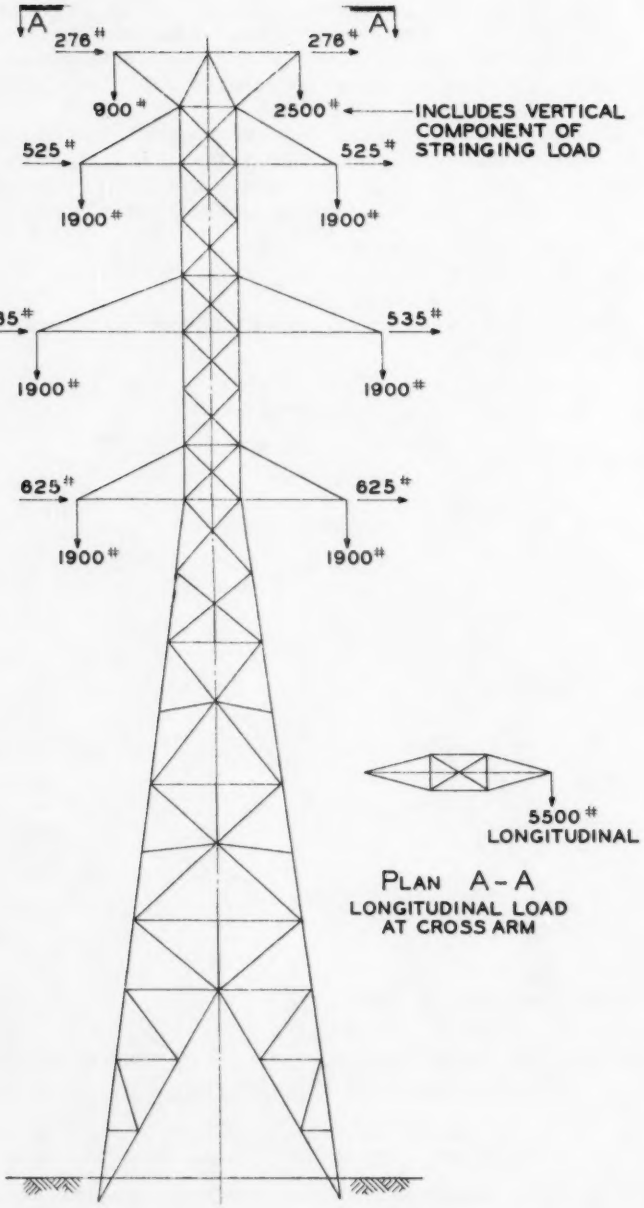


FIG. 12.—STRINGING LOADS

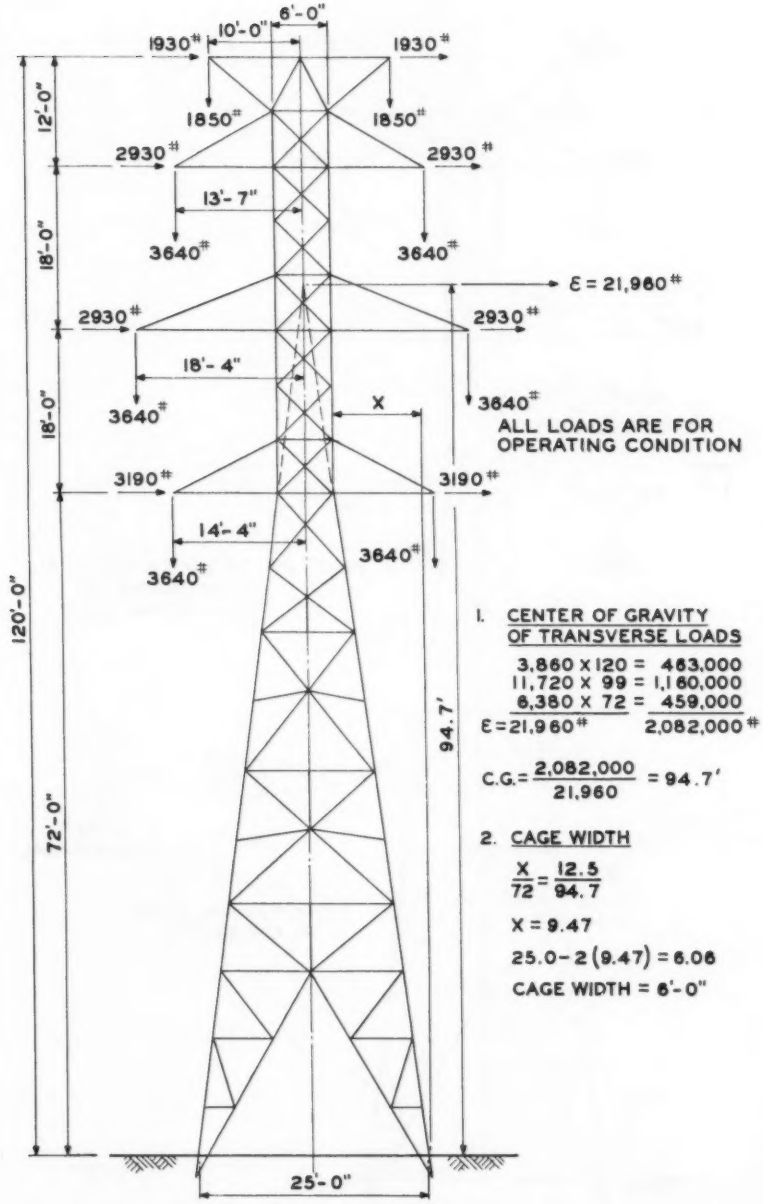


FIG. 13.—LEG INTERSECTION AT CENTER OF GRAVITY OF TRANSVERSE LOADS

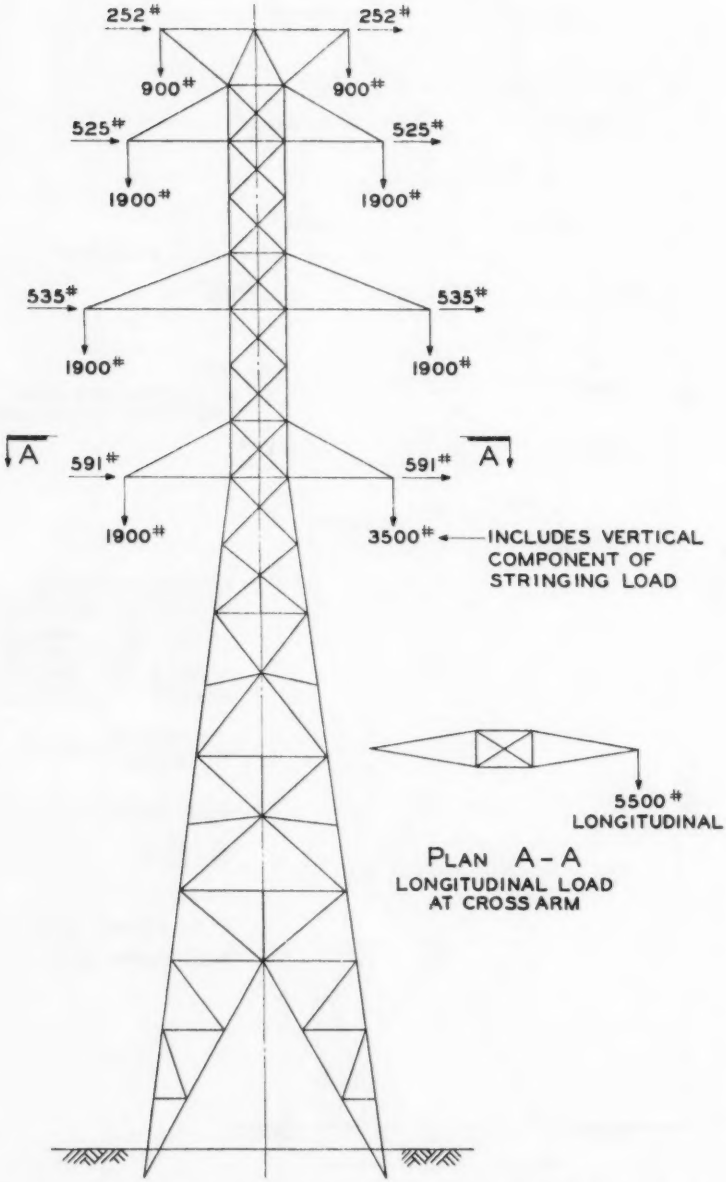


FIG. 14.—LOGITUDINAL STRINGING PULL AT BOTTOM CROSS ARM

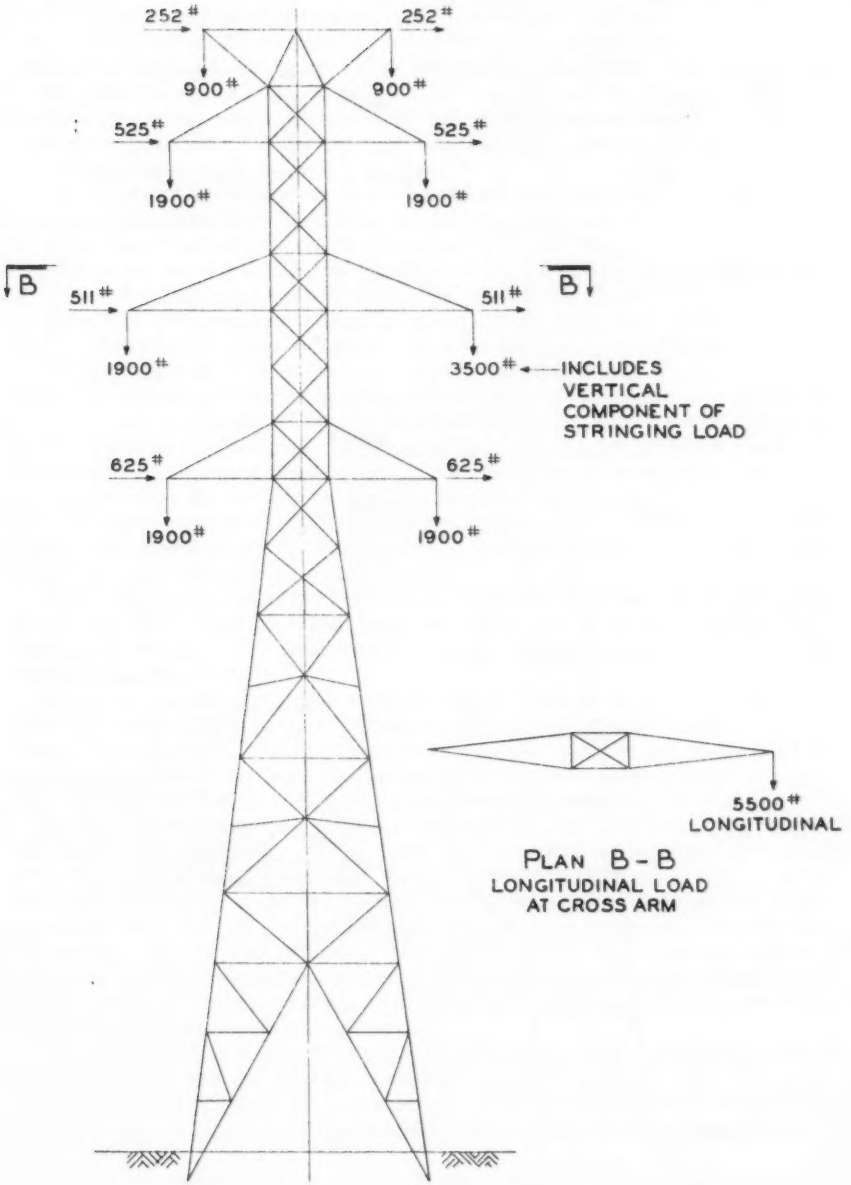


FIG. 15.—LONGITUDINAL STRINGING PULL AT MIDDLE CROSS ARM

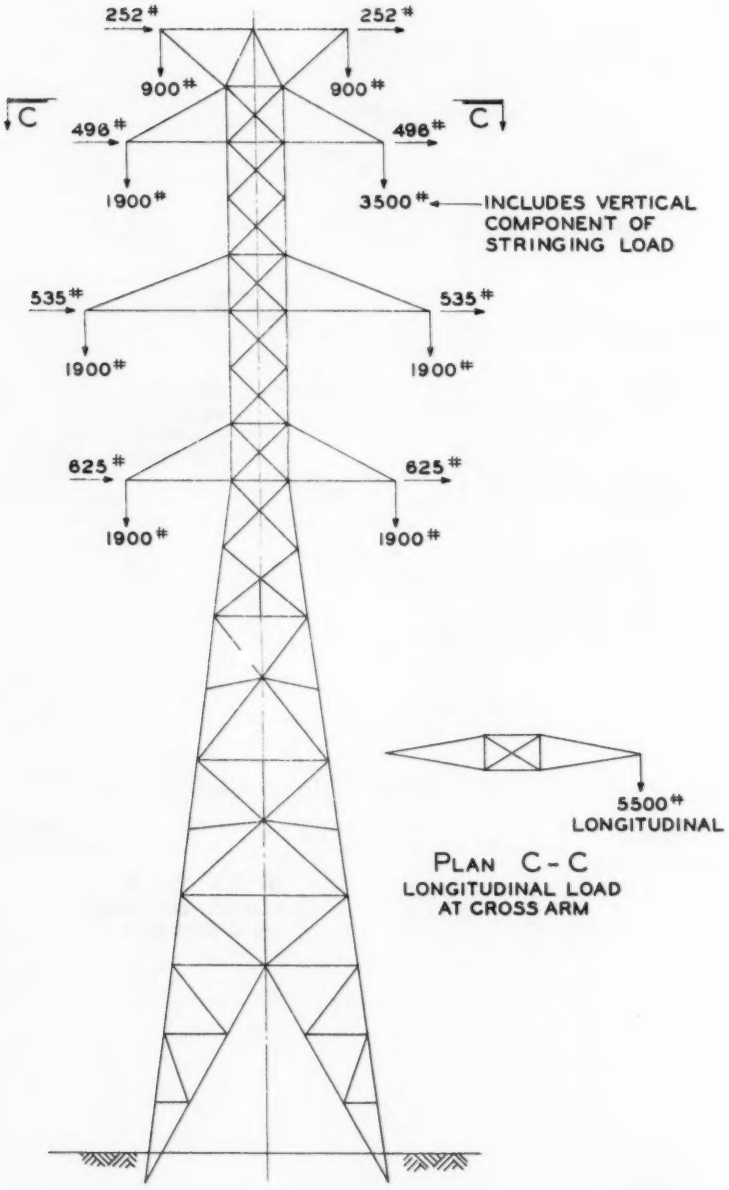


FIG. 16.—LONGITUDINAL STRINGING PULL AT TOP CONDUCTOR CROSS ARM

is required only during the wire stringing operations and these occur under reduced wind, reduced wire tension, and reduced vertical load.

Fig. 17 shows the heavy vertical condition on the cross arms. This loading condition is applied to each cross arm with no other load acting and consists of the weight of the wires covered with 1 in. of vertical ice. Maximum cross-arm stresses can occur under either the heavy vertical or stringing conditions. Customary practice calls for use of a square cross section with all faces of the tower identical.

It is apparent from these loading conditions that the tower need not be as strong in the longitudinal direction as in the transverse direction and after considerable investigation, the shape shown in Fig. 1 was selected, utilizing a rectangular cross section for the tower cage and base as being the most economical. The acceptance of a rectangular cross section increased the complexity of the design for torsional loads upon the tower and perhaps illustrated the reason why the rectangular cross section is not more commonly used.

A basic height of 72 ft to the lower cross arm was selected. Comparisons were made between a vertical spacing of cross arms of 20 ft with the conductors in line vertically against a staggered vertical alignment, with a vertical spacing of 18 ft between cross arms. After careful study, the latter scheme was selected as being the most economical. The reduction in height resulting from this change reduced the load on the legs considerably, but the increase in torsion due to the extended center cross arm nullified this to some degree. This would have been a major disadvantage on a line containing many angle towers but did not have a major effect on this line.

Wind acting on ice-covered conductors will set them to dancing or galloping, and if two wires come too close together, there will be a flash between them causing an outage. For this reason, it is extremely important that proper clearance be maintained.

In the transverse direction, a base width of 25 ft was selected as being the most practical. This gave an excellent compromise for the problem of leg size versus lacing length. As the tower base spreads out, the leg loads become smaller, but the lacing members becomes longer. Somewhere, there is a balance between the two, and 25 ft was calculated to be the most desirable.

In the transverse direction a cage width of 6 ft was selected on the following basis: The intersection of the tower legs was fixed at the center of gravity of all the transverse loads so that the legs below the cage would take all of the load with nothing going to the lacing. Since the legs are heavier and stiffer members, they can take the transverse load better and more economically than the lacing. Fig. 13 shows the leg intersection. The 11 ft base dimension in the longitudinal direction was selected by trial to give the same stress in the legs under the stringing condition as for the transverse heavy loading and, thus, utilize the leg material most efficiently.

The cage width of 4 ft in the longitudinal direction was essentially selected by the same reasoning. The ideal width, giving equal leg loads under the transverse heavy loading, and longitudinal stringing conditions, would have been 3.3 ft, but since this is impractical from a detailing standpoint, it was increased to 4 ft.

Pattern of Bracing.—Selection of the cross bracing in the cage was limited to one choice since the 18 ft spacing between cross arms divided nicely into 3 spaces giving 45° bracing in the wide face and approximately 56° in the narrow face. Since this bracing is of relatively short length, 8 ft-5-1/2 in. in the wide face and 7 ft-0-7/8 in. in the narrow face, the members were designed to take

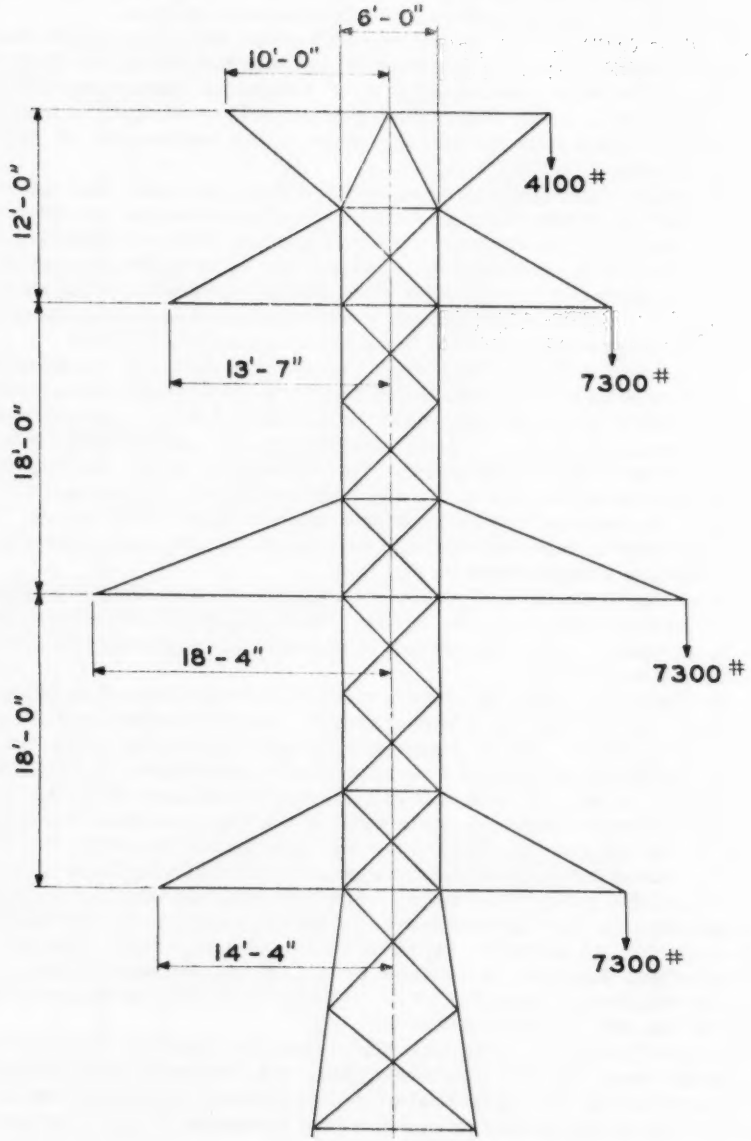


FIG. 17.—HEAVY LOADING CONDITION ON THE CROSS ARMS

either compression or tension. This compression system of bracing was carried down into the wide face of the tower body for two panels and at that point a tension system was used in which the members were designed to act only in tension. By trial it was found that the tension system, with one member acting and the other redundant, was more economical at this point than the compression system.

In the narrow face of the tower, the compression system of bracing was found to be the most economical throughout and for that reason it was carried down to the junction between the tower body and the legs and a tension system was used for the legs.

Redundants were placed at the most advantageous position to reduce l/r in other members so that their weight was more than compensated for by the saving in the member braced.

Allowable Stresses and Other Design Criteria Used.—The following formulae were used for design of compression members using A-7 steel:

$$f_c = 33{,}000 - 130\ l/r \text{ for } l/r \text{ up to } 150 \quad \ldots\ldots\ldots \quad (14a)$$

and

$$f_c = 28{,}000 - 100\ l/r \text{ for } l/r \text{ 151 to } 200 \quad \ldots\ldots\ldots \quad (14b)$$

with a maximum of 30,000 psi. This is based on the AISC compression formulae for other structures with an increase in the allowable stress by the same ratio that the allowable tensile stresses for towers are increased over the allowable tensile stresses for buildings. The high strength steel formula used, were Eqs. 9 through 13.

Maximum l/r in tension members was established at 500 and allowable tension stress was set at 33,000 psi, for A-7 steel, and 50,000 psi for high strength steel. These values are the theoretical yield points of A-7 steel and high strength steel respectively. It was recognized that it is reasonable to expect that steel received from the mill will test to a yield point considerably above the minimum. It was felt, however, that under the present design criteria, which ignores eccentricities and the effect of connecting angles by one leg only, this margin was needed as compensation.

Ultimate bolt shear and bearing values were taken from ASTM Specification 394 for transmission tower bolts and are: 45,000 psi on net section for shear, and 90,000 psi on net section for bearing.

Stress Analysis.—Stresses in the tower are caused by the stringing pull, transverse force due to angular wire tension, wind, and vertical load. It was assumed that the vertical loads were carried by the four tower legs. Transverse loading on the tower is equal to the transverse component of tension in all wires plus wind on the wires and tower.

Forces on Tower due to Wire Tension and Wind.—Stresses due to torsional forces P and T (Fig. 18) were determined by the method of least work (commonly known as Castigliano's second theorem). The theorem states that the total internal work performed by the fiber stresses in a statically indeterminate structure is that which is the minimum consistent with equilibrium. In this solution, the horizontal cross bracing members in the tower at the elevation at which the load was applied through the cross arm were treated as redundant members. (The term "redundant member" applies to the usual terminology for indeterminate structures.)

Effect of horizontal cross bracing at other levels was ignored. Thus, in the example shown in Fig. 19, members AC and BD were removed as redundant members and the remainder of the tower was treated as the base structure and analyzed for the stresses caused by the loads applied to the base structures, plus the unit loads S, as shown in Fig. 19 and in column stress = F.

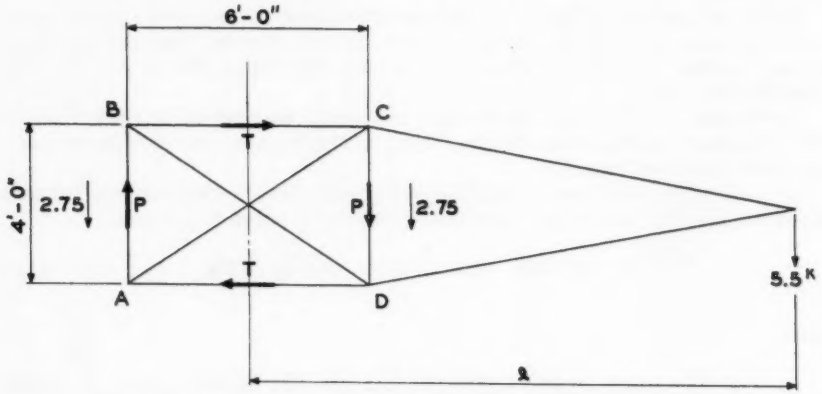


FIG. 18.—TORSION IN TOWER CAGE DUE TO STRINGING LOADS

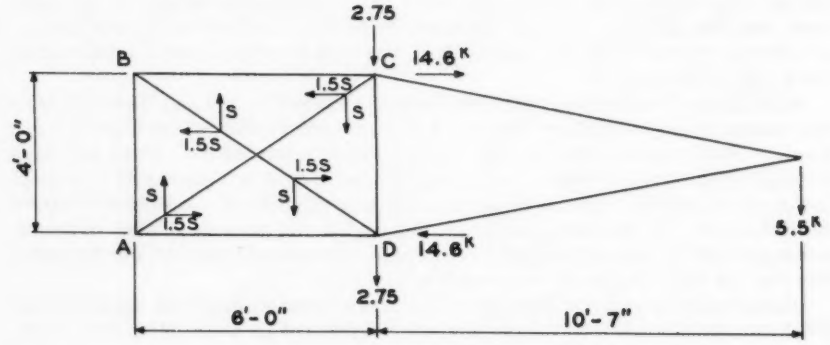


FIG. 19.—UNIT STRESSES IN REDUNDANTS

Work on any truss member equals the maximum deformation multiplied by the average magnitude of stress.

$$W = \frac{F\,L}{A\,E} \times \frac{F}{2} = \frac{F^2\,L}{2\,A\,E} \quad \ldots\ldots\ldots\ldots\ldots\ldots (15)$$

$$\text{the total internal work} = \Sigma\,\frac{F^2\,L}{2\,A\,E} \quad \ldots\ldots\ldots\ldots\ldots (16)$$

In order to get a minimum, the equation if differentiated with respect to the unknown S, set equal to zero and solved is

$$\frac{dW}{dS} = \Sigma\,(F)\left(\frac{L}{A\,E}\right)\frac{dF}{dS} \quad \ldots\ldots\ldots\ldots\ldots\ldots (17)$$

All of the stresses were entered in a table (Table 2). All of the terms in the $F \times \frac{dF}{dS} \times \frac{L}{A}$ column were added algebraically, and, setting the expression equal to zero, a value of S was obtained. This was substituted for S in the Stress = F column giving the true stress in each member.

TABLE 2.—STRESS COMPUTATIONS

Member (Fig. 16)	L	A	L/A	Stress = F	$1 \frac{dF}{dS}$	$F \frac{dF}{dS} \frac{L}{A}$	Torsional Stress
1	8.46'	.62	13.6	-10.3 + 2.11S	+ 2.11	-295 + 60.3S	-6.6k
2	6'	1.69	3.55	-50.8 + 10.5S	+ 10.5	-1890 + 392S	-32.3k

E being constant is eliminated

$$\Sigma = -35217 + 20{,}023S = 0$$

$$S = \frac{3517}{20{,}023} = 1.76k$$

L = Length in feet
A = Area in square inches
Stress in Kips

Redundant Stress = (1.8) (1.76) = 3.2k

Stresses caused by angular pull due to wire tension and wind were added to the torsional stresses in the members in the wide face of the tower to produce the maximum. Leg stresses were obtained for the operating condition by statics and the maximum stress was used in sizing the members. The final maximum stress combination for each member is shown in Fig. 20, for which all stresses are indicated in kips and + indicates tension and - tension. Where members are either in tension or compression only the governing load is shown.

Review of Completed Tower Design.—The saving in weight and cost for this tower, as compared to previous designs, was 32% and 37%, respectively. The design used previously weighed 18,180 lbs as compared to 12,306 lbs for this tower. This saving was brought about by designing for a stringing load of 5,000 lbs instead of a broken wire, by careful check of design criteria and using high strength steel were advantageous, and by utilizing an efficient rectangular shape and by obtaining bids from many fabricators, including those who were not tower designers. In a 100 mile line, this saving represented approximately $500,000.

In evaluating the erection bids, it was noted that there was no apparent increase in cost per pound due to the "lightness" of the tower. The erection of these towers proceeded satisfactorily.

Testing of Tower.—Normally, transmission towers are not tested. However, this tower is not a conventional design and because it contained a new approach to the distribution of torsional stresses, together with a rectangular configuration, it was recommended that a test be made.

Two multispan steel frames, one parallel and one at right angles to the tower, were used to provide the anchorages for the transverse and longitudinal pulls. Figs. 21, 22, and 23 show the tower and the rigging used for the vertical and horizontal loads. Loads were applied in increments of 50%, 75%, 87.5% and 100% of total load.

Vertical loads were applied by suspending trays filled with steel slab weights. Transverse and longitudinal loads were applied by an electric winch with cables going to single, double, or triple trees. Each increment of load was held for 5 min and both the longitudinal and transverse deflections were shot with a transit.

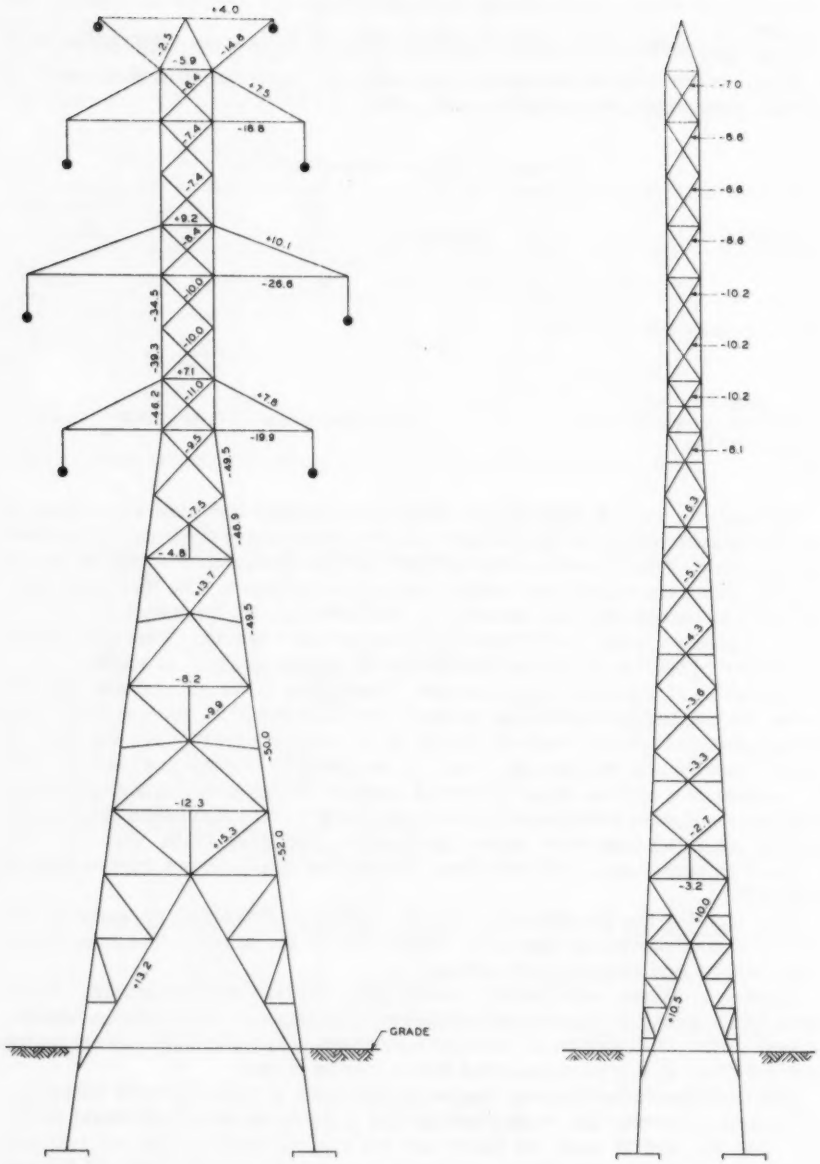
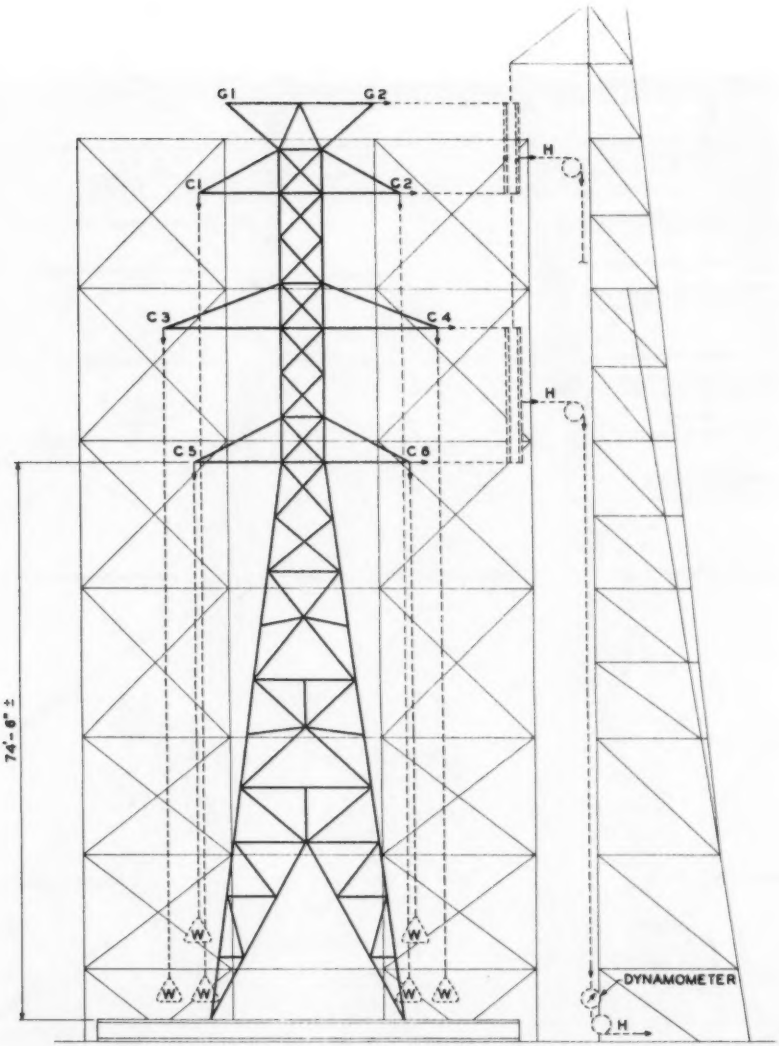


FIG. 20.—MAXIMUM STRESSES IN TOWER MEMBERS



1. TEST STRUCTURES SHOWN IN LIGHT LINES.
2. TOWER SHOWN IN HEAVY LINES.
3. RIGGING SHOWN IN DOTTED LINES.
4. VERTICAL LOADS "W" APPLIED WITH SUSPENDED TRAYS LOADED WITH 80 M PLATES PLUS WT. OF TRAY & HOIST.
5. HORIZONTAL LOADS "H" APPLIED THROUGH MOTOR OPERATED DRUM.

FIG. 21.—RIGGING USED FOR VERTICAL AND HORIZONTAL LOADS

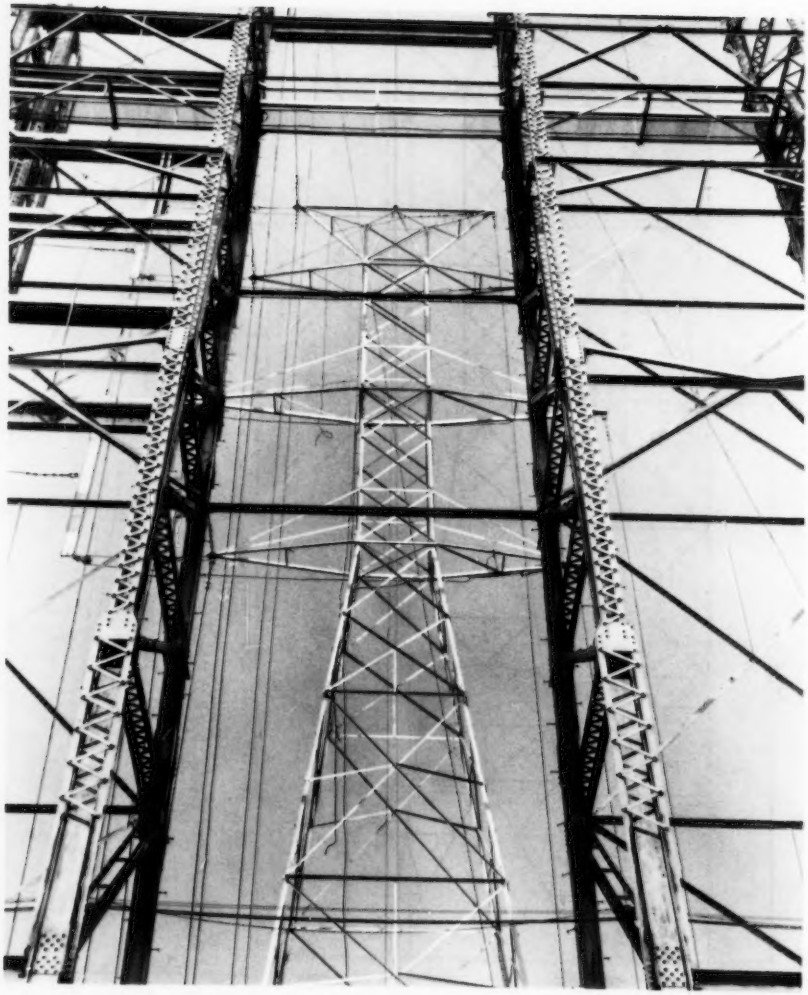


FIG. 22.—RIGGING FOR VERTICAL AND HORIZONTAL LOADS

FIG. 23.—RIGGING FOR VERTICAL AND HORIZONTAL LOADS

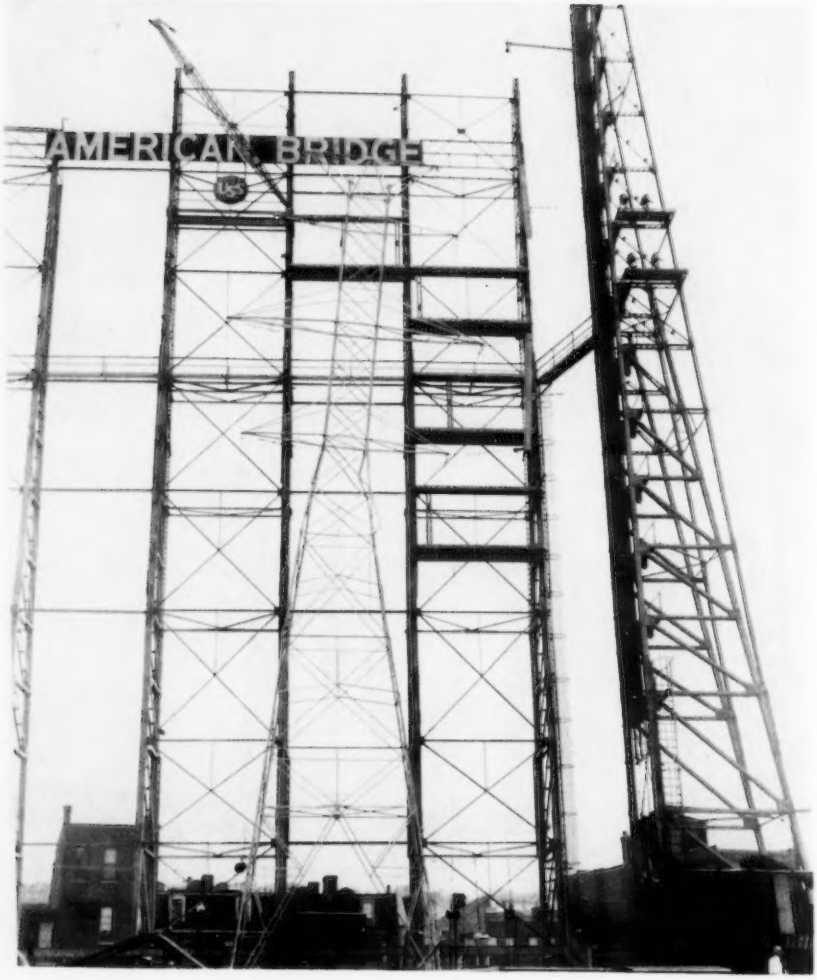


FIG. 24.—FAILURE OF TOWER LEGS BY BUCKLING DURING TEST

After the completion of the tests for the operating loads, the vertical loads were held constant and the transverse load was increased in increments of 1,000 lb. The tower failed dramatically in the compression legs just below the bottom cross arm, by buckling as shown in Fig. 24. Measured deflection at the time of failure was 21 in. at the top of the tower and 6 in. at the bottom of the cage.

The tower successfully passed all tests and failed at 12.4% over the specified operating load. What does this mean? Does it mean that the tower was over designed by 12.4%? Does it mean that the yield strength of the materials was more than the values used in design?

Answering these questions, the failure load in the member was 54.4 kips. It was designed for a capacity of 52.8 kips using a yield point of 50,000 psi. However mill tests on the steel revealed a yield point of 55,000 psi which would give an allowable stress of 54.5 kips per sq in. The angle chosen was a 3-1/2 by 3-1/2 by 1/4, high strength and came as close as possible to being the ideal section.

Yield strength higher than specified is worthwhile in the low ranges of l/r. In the high ranges of l/r, the allowable compressive values lie within a very narrow band and may be governed by the critical buckling stress.

Some transmission engineers advocate using a 36,000 psi yield in view of favorable mill reports, but this is not recommended here. Perhaps the cushion between the 33,000 and 36,000 yield is a built in safety factor to compensate for the many deviations from the normal found in tower design, such as lap splices and unconnected angle legs. If design based on a higher yield is proposed, then too must be considered moments in lap splices and reduction in area for angles connected by only one leg. Also, to be considered is the fact that in the $l/r = 140$ range, buckling governs since it is a function of shape and length and is independent of the yield point of the material (Fig. 9).

CONCLUSIONS

It is hoped that this paper will create enough interest to stimulate a discussion on the design criteria presented and that it will result in improvements of existing codes so that eventually all towers can be designed on a comparable basis throughout the country.

Items of importance to be considered in the design of an economical transmission line are as follows:

1. Selection of design loading conditions comparable with the anticipated loads rather than provision for every remote contingency;
2. Willingness to consider a rectangular tower cross section with its increased complexities;
3. Extensive studies on various configuration to achieve the optimum strength with the least material;
4. Careful checking and rechecking of members to insure selection of minimum sizes of use of a computer program to select them automatically; and
5. Incorporation of high strength steel in the design at advantageous points.

ACKNOWLEDGMENT

This paper is based on the experiences gained by the firm of Sargent and Lundy in making a review of all the factors affecting the cost of transmission

towers, under the auspices of the Public Service Company of Indiana. The tower was tested using the facilities of the American Bridge Co., Pittsburgh, Pa.

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Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

REGLA STEAM ELECTRIC STATION

By George T. Ingalls,[1] M. ASCE

SYNOPSIS

This paper describes the design of a four unit steam electric station. Unusual features include design of foundations to cope with stiff clay, rock strata and slag deposits, outdoor-type construction, strengthening gantry crane and ventilating louvres for turbo-generator housings.

INTRODUCTION

The Regla Steam Electric Station is located in the town of Regla, on the eastern shore of Havana Harbor, in Cuba. The site had been used as a recreation field (see Fig. 1). The lot is approximately 430 ft by 440 ft or about 4 1/2 acres. It is bounded by the waters of Havana Harbor on the west and north side.

The town of Regla was a haven for pirates in colonial days and any sailing ships that docked there were subject to raids. It was quite common for these ships to be stripped of their copper bottoms.

The site was previously occupied by a large iron foundry. There were a number of slag deposits buried in the ground and, also, around the shore line. Just off the west shore was a submerged marine railway.

A number of sites were investigated before Regla was finally selected. No site has all the ideal conditions such as water supply, location near large power users, good sub soil for foundations, general system requirements, land values,

Note.—Discussion open until November 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. PO 3, June, 1960.

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transportation, and a good supply of labor. The Regla site, however, was ideal as to situation with adequate cooling water, good supply of labor, and good transportation (a ferry line is adjacent to the plant within 10 minutes commuting distance to Havana). The main drawback was its restriction as to size of lot. It required considerable juggling (See Fig. 2) to get four units with an ultimate capacity of 209,000 kw and all auxiliaries on 4 1/2 acres. This includes substation, oil tanks, office building, machine shop, chemical storage, and laboratory building. In addition to this, the Cuban government required a 20 ft clear right of way on the harbor side for emergency vehicles. This is called the "Zone of Vigilance."

The location has the advantage of being accessible for barges to unload fuel oil directly into the two 15,000-barrel storage tanks. In 1958, it was suggested that high viscosity oil be used and pumped from storage tanks about two miles

FIG. 1.—PLANT SITE BEFORE CONSTRUCTION

away. This oil is the residue of the refinery and requires a high temperature for liquid flow.

DESCRIPTION OF PLANT

The Regla Steam Electric Station is of the "semi-outdoor" type, composed of steel frame and concrete floors. The turbine and auxiliary bays have no side walls. The turbo-generator is completely housed on the operating floor

and a 50-ton gantry crane straddles this housing to service the turbo-generator through removable hatch covers on the roof. The oil-burning boilers are an outdoor installation (two boilers for each unit) along with their fans and auxiliary equipment. The remaining structures consist of a combined office and electrical building, machine shop, laboratory, and chemical storage building. All of these buildings are of reinforced concrete with flush brick spandrel walls, cement coated.

Comparative costs were made of a closed station (walls, roof and overhead crane) and a semi-outdoor station (no walls or roof, except housing for turbo-generator and gantry crane). There was an estimated savings in excess of 1/2 million dollars using the semi-outdoor station. Total cost of Units 1 and 2, including substation, are $6,600,000 and $6,730,000, respectively.

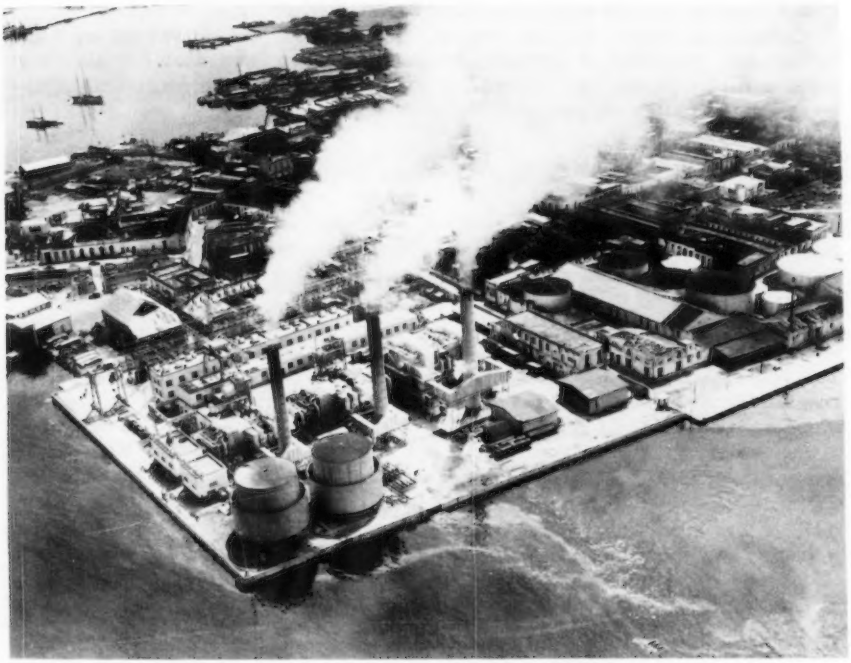

FIG. 2.—FINISHED PLANT—UNITS 1, 2 AND 3 OPERATING

Service water is supplied from the municipal water supply to a 640,000 gal reinforced concrete storage tank, placed underground. The storage tank was necessary due to the uncertainty of a continuous supply of city water. Water is demineralized before being used as boiler make-up. The storage tank is compartmented for separate storage of potable water. Circulating water screen chamber and intake canal is located on the north side of the plant while the discharge canal is on the west side.

To retain the fill on the north and west boundaries of the site, a sea wall of interlocking steel sheet piling was driven to a depth of 40 ft to 50 ft with tie

backs 15 ft on centers, consisting of four 1 1/4 sq in. tie rods encased in a 12 sq in. concrete envelope. These were anchored back to a concrete dead man supported by two steel pipe batter piles. The sheet piles were capped with concrete which extended 2 ft below mean low water. A retaining wall constructed on top of the cap retains yard fill. The pile cap serves as a waler and as protection for piles above mean low water.

The switchyard and transformers are adjacent to the electrical building on the east side. All major equipment of the switchyard, including the steel structure, is supported by piles. Transformers rest on rails which extend to the unloading bay.

The highest variation of tide on record in Havana Harbor is about 2 ft or el 102. Finished yard grade of el 107 ft, gives a margin of 5 ft.

The plant area on the street sides (south and east side) is enclosed with Cuban brick plastered wall topped with barbwire barrier.

Cathodic protection for the steel piling was discussed but was not used due to the cost. However, a complete grounding system was used for all sheet steel and pipe piling. All structural steel, mechanical and electrical equipment above grade were grounded.

EQUIPMENT HANDLING

The turbo-generators for Units 1 and 2 were unloaded at the dock in Havana, placed on railroad flat cars, and brought to the site. The bridges had to be shored, to carry the increased load. At a sharp curve a corner of a one-story building was cut off to give the necessary clearance.

For Unit 3, the authorities at the Havana unloading dock would not permit the generator stator which weighed 118 1/2 tons, to be unloaded from a boat. The nearest dock that could handle this load was at New Orleans, La. Arrangements were made to have a converted L.S.T. landing barge at the dock when the boat arrived from Germany. A low-boy truck was placed on the barge to receive the stator. A floating derrick made the transfer, (See Fig. 3). In order to unload from the L.S.T. boat at the Regla site, a portion of the sea wall was cut down to allow the ramp of the L.S.T. to seat on the ground. When the boat was securely tied to anchor posts, the low-boy truck was hauled to the unloading bay of the turbine building. The gantry crane lifted and placed it in position on the turbine pedestal. The remainder of the turbo-generator was brought to Havana by the overseas boat, unloaded at the dock, and trucked to the Regla site.

INTAKE STRUCTURE

The intake structure consists of screen chamber, divided into four sections. A single 4-unit intake canal is open for the major portion and is slabbed over at the circulating water pumps. The entire structure is of reinforced concrete construction supported on wood and steel piles. The screen chamber has a concrete skimmer wall at the entrance extending 3 ft below mean low water, stop log guides for the reinforced concrete stop logs. Following these are coarse and revolving screens. When the revolving screens are removed for servicing a set of fine screens are placed in the fine screen guides.

The canal was designed as a rigid frame structure with concrete walers and struts near the top.

A 20-ton gantry crane services the revolving screens. The crane rail is carried on a concrete girder runway extending 54 ft on the easterly side of the screen chamber. When construction started on the intake chamber, excavated material was used to form an earth dyke. Due to flood tide conditions during

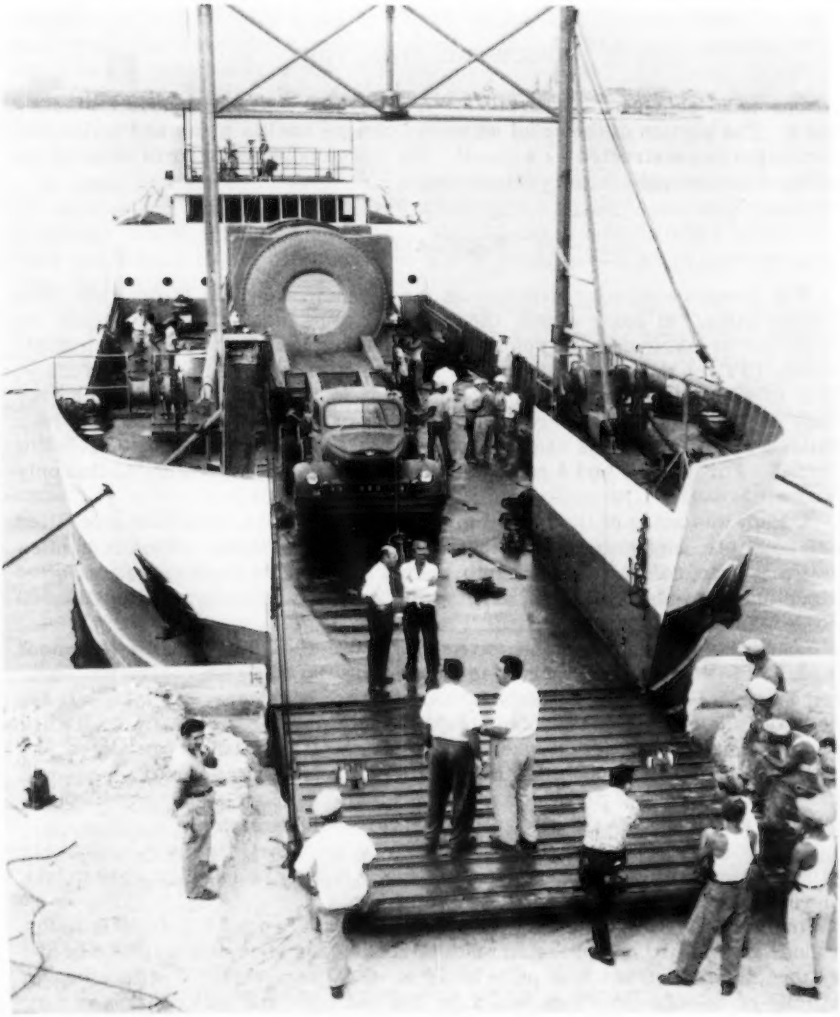

FIG. 3.—UNLOADING STATOR FOR UNIT 3

construction, the dyke collapsed at the harbor entrance and flooded the work area. Sheet piling was then driven at the entrance and after construction was completed, it was cut down to the invert of the chamber. The invert of the screen chamber is el 88 ft, and the invert of the canal is el 90 ft. The top of

the deck slab is at el 107 ft. The intake canal decreases in width from 20 ft at Unit 1, to 8 ft at Unit 4.

Due to the close proximity of the foundation for the electrical building to the intake canal, it was necessary to build the intake canal before driving piles for the electrical building.

DISCHARGE STRUCTURE

One canal services Units 1 and 2. A separate canal is provided for Units 3 and 4. The portion of the canal under the turbine auxiliary bay and boiler mat foundation is constructed as a tunnel. The tunnel portion supports some of the boiler columns and auxiliary equipment.

FOUNDATIONS

The presence of slag deposits in the ground presented difficulties when driving piles. In some cases, the slag had to be blasted if the chopping bit could not break it up. A typical boring for the Unit 1 site showed approximately 10 ft of dump fill and silt, 10 ft to 25 ft of gray sandy clay and muck, 25 ft to 40 ft of yellow sandy clay, and 40 ft to 100 ft of hard compact gray clay. In the Unit 1 area there was little indication of gray or serpentine rock while borings for Unit 2 showed solid serpentine rock 16 ft to 40 ft below ground. For Units 3 and 4 rock was 30 ft to 60 ft below ground and that only in small areas.

A study was made of the type of pile to be used. Wood, steel tube pipe filled with concrete, and precast concrete piles were considered. Wood test piles were driven to refusal at 24-ft depth, with 300 blows for the last foot; and steel tube piles, closed-ended, with steel point reached refusal at 27 ft, with 500 blows for the last foot.

Load tests for the wood piles gave a capacity of 40 tons or a total settlement of 11/32 in. and the steel tube piles gave a capacity of 80 tons for a total settlement of 7/32 in. With a safety factor of 2, a design capacity of 20 tons for the wood pile and 40 tons for the steel tube pile was selected. It was decided to use untreated wood piles where the pile cut-off was below low water and 10 3/4 in. O.D. steel tube piles filled with concrete and reinforced full length where pile cut-off was above low water. The pile reinforcing (longitudinal and tie bars) was placed in the pipe piles to secure column action.

When construction started wood piles could not penetrate the compact clay without brooming. As a result, ten-3/4 in. O.D. steel tube piles were driven where wood piles were specified.

For Unit 2 installation, the soil borings indicated rock 16 ft to 40 ft below ground. Unconfined compression tests of rock cores showed a variation of 398 psi to 4,295 psi. Steel tube piles of 14 in. O.D. were selected with a design capacity of 30 tons (this was based on 400 psi over the pile cross section). Further studies were made substituting caissons in place of a group of steel tube piles. Four feet was a minimum diameter to provide adequate working space. A 4-ft caisson would be equal to 11 piles in bearing capacity and would provide considerable savings over the cost of piles. An inspection of the profile of the serpentine rock showed considerable slope in a number of cases. In view of this, it was specified that caissons should be seated in the rock to get a level bearing. If the bearing value of the rock was less than 400 psi the

caisson was to be belled out at the bottom. Where loads were spread out or much less than the capacity of the minimum size caisson, steel tube piles filled with concrete and reinforced full length were used. All steel pipe piles were closed-ended with cast steel points.

It was not economical to use caissons for Units 3 and 4, as the difficulties encountered, such as excavating in a confined space at greater depth, keeping excavation dry (water depth would be about 30 ft to 50 ft) cost of pumping, etc., would more than offset the cost of piles.

DESIGN CRITERIA

A study of the Cuban weather records showed wind velocities as high as 150 mph actual. In accordance with the recently revised "American Standard Building Code Requirements for Minimum Design Loads," (A58.1-1955), the wind pressure formula at a height of 100 ft would give a much greater value than the 50 lb which was used.

All structures were designed for a wind pressure of 50 psf on the projected area. Past experience has indicated that this is ample for generating stations in Cuba. Lesser wind pressures are used in our other Latin American countries. To provide for this lateral force at the foundations, batter piles were used. This applied to all structures where the ratio of height to width was less than one, or where the lateral force was greater than 1,000 lb per pile.

VENTILATION OF TURBINE HOUSING

The turbine housing consisted of a steel frame, concrete roof and corrugated asbestos siding. A study was made for ventilating the enclosure by means of vertical louvers and movable shutters. It was desirable to bring air in on the sides just above the floor and exhaust it through eight power ventilators in the roof for each unit. Stationary windows, without vents, were provided on the sides so that there would not be any short circuiting of air over the turbogenerator. A unique sliding shutter was provided to close off the vertical louvers in the event of a driving rain. These louvers consisted of a series of asbestos cable trays on the outside and another series on the inside offset half the width so that any rain laden air coming in would have to make two passes before getting inside the housing. If there was much moisture in the air, shutters would seal it off and it would drop down and drain to the outside. This arrangement is shown in Figs. 4 and 5. The ventilation was designed for one air change a minute. Note the vertical louvers under the windows in Fig. 5.

STRENGTHENING GANTRY CRANE

A Colby, 50-ton traveling gantry crane with a 10-ton outrigger and a 10-ton auxiliary hook was placed on the operating floor, which is the same as the roof level. The 10-ton outrigger services the circulating water pumps and the heaters. The generator stator for Unit 1 weighed 58 tons. The Unit 2 generator stator weighed 92 tons. To handle this overload, it was necessary to pick up the load from the main drum through a needle beam and distribute it through sheaves and cable to each end of the crane bridge. This brought half of the

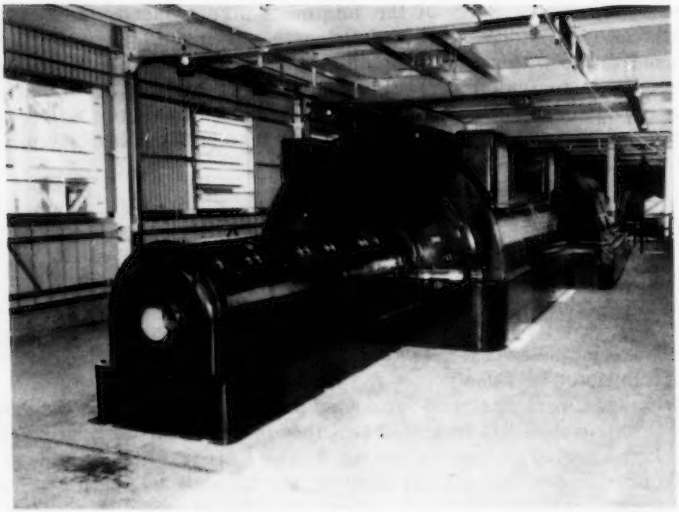

FIG. 4.—INTERIOR VIEW OF TURBINE HOUSING

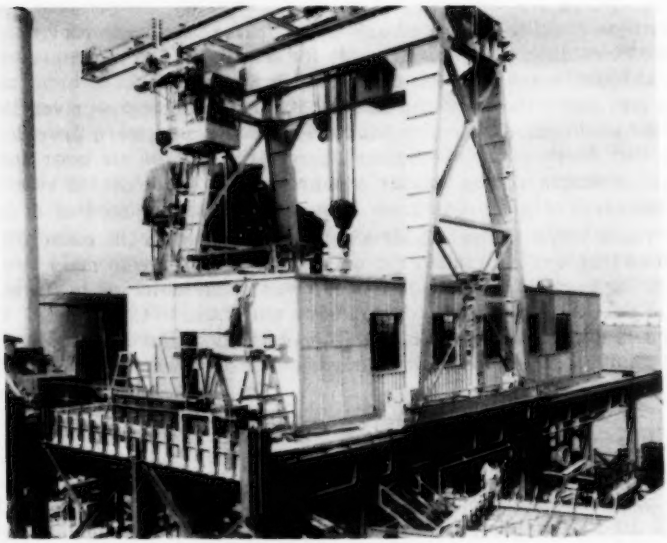

FIG. 5.—CRANE BEFORE STRENGTHENING

load directly into each gantry leg. Heavier cable replaced the existing and additional stiffeners, and bracing was installed (see Fig. 6). The maximum wheel load of a crane is determined by placing the lifted load at one end of the crane bridge. With the arrangement that has been outlined, the increased original wheel load was 18%. A check was made of the stresses in the steel framing and foundations for the weight of stator for Units 3 and 4 and they were found within the safe limits.

SUBSTATION

It has been general practice in substations to place the transformers on rails so that the transformer can be readily moved when repairs are needed.

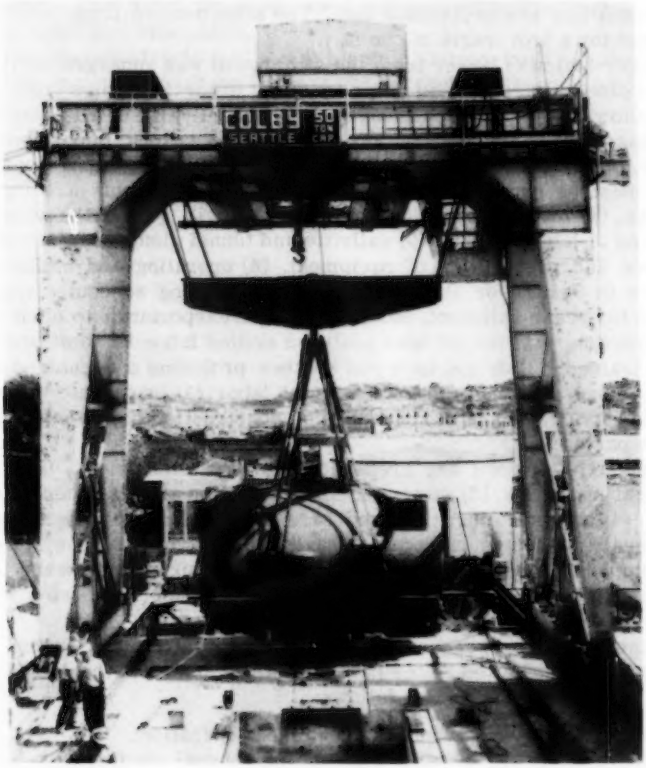

FIG. 6.—50 TON CRANE

Where the substation is adjacent to the generator plant, the crane runway is extended one bay beyond the main building for servicing transformers and other equipment. At Regla this extension is adjacent to the machine shop. When a substation is away from the generating plant a transformer tower is erected on the site and the transformer is rolled underneath and serviced by an electric hoist. All oil filled transformers are separated by reinforced con-

crete fire walls. Crushed stone was placed around the transformers for oil drainage in the event of fire. The depth of this crushed stone was based on releasing the total quantity of oil through 50% voids in the stone.

A study was made for reducing the cost of formwork and standardizing the size of transformer fire walls. Types considered were (a) reinforced concrete piers with precast concrete panels, (b) reinforced concrete piers with concrete block filled with concrete and reinforced, (c) battered, reinforced concrete wall with a varying base dimension and 8 in. at the top, and (d) precast concrete—post tensioned.

It was found that type (c) was the most practicable and economical, if the size of transformer walls were standardized. By using the manufactured steel frame plywood forms in 2-ft widths, with varying lengths, clamps and ties, there was a saving in the cost of formwork, depending on the number of reuses. A standard firewall drawing, showing wall thinkness and reinforcing, was made for all sizes that are in common use. The sizes ranged from 14 ft to 28 ft in height, and for a unit length of one ft.

The distribution of power from the switchyard was underground through oil pressure pipe to the overhead Regla terminal tower two miles from the plant.

The following are typical, although not necessarily true of all Latin America.

Purchasing and Shipping.—(1) Long delays in obtaining import permits, (2) delays in shipping due to suppliers not equipped to handle export material, (3) packing must be exceptionally substantial where shipments are lightered to port docks, (4) lifting facilities at port of entry will govern the weight and/or dimensions of largest piece, (5) railroad and tunnel clearances will determine fabrication and preparation of equipment, (6) operating and maintenance instructions in Spanish or Portuguese, and (7) shipping schedules require long lead time for ocean shipment, local customs, transportation to plant site.

Construction.—(1) Do not have qualified skilled labor in some areas—labor must be trained, (2) do not have the weather problems encountered with subzero conditions, (3) language barriers with labor, (4) inadequate telephone and telegraph service, and (5) maintenance of construction equipment—difficult to get spare parts.

Design.—(1) Drawings are prepared in metric dimensions, (2) limited by use of local materials, (3) restricted sizes of rolled shapes necessitate welded sections, (4) only smooth reinforced bars can be obtained in some countries, (5) poor quality concrete aggregates—have to use lower stresses, (6) unable to obtain complete weather data, (7) do not have to design for snow and ice loads, and (8) architectural treatment, both inside and outside, is much simpler and less expensive.

In some respects, concerning design and construction, Steam Electric Stations built in the United States have similar problems to those built in Latin America. Some of these are as follows:

1. Arrangement of equipment for economical operation;
2. operating loads are the same, except for unusual conditions;
3. obtaining adequate circulating water for condenser cooling;
4. fuel sources, shipping, and storage facilities;
5. location of plant on ground above maximum probable floods;
6. location of plant on the best available soil conditions;
7. location of plant for service by railroad, highway or river navigation;
8. location of plant and switchyard for outgoing transmission lines;
9. adequate ventilation of plant; and
10. sufficient room for construction and for future expansion.

CONCLUSIONS

When the Regla Steam Electric Station was planned, the demand for power was far greater than the supply. One of the oldest stations, the Consolidated Steam Electric Station which is across the harbor, (in Havana), needed major overhauling. It was planned to do this work when Regla Unit 1 was completed. The demand, however, was even greater than expected and so Unit 2 was started and following this was Unit 3. During the last 10 years, in the Havana area, a large number of apartments, office buildings, residences, and some hotels have been built. Greater use of air conditioning, electrical appliances, and industrial expansion have increased the load and boosted the economy of the country.

The outdoor-type of construction seems ideal for ventilation. However, when Unit 1 was completed, the operators complained of getting wet during driving rains. Two sides were enclosed with corrugated asbestos on steel frame with ventilated steel sash.

The design and construction of steam electric stations built in Latin America present some unusual problems that are not encountered in the United States.

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ROCKFILL DAMS: KENNEY AND CHEAKAMUS DAMS[a]

Closure by William G. Huber

WILLIAM G. HUBER,[1] M. ASCE.—The discussion by Mr. Cooke on Kenney Dam and on the movements within a rockfill, and the discussion by Mr. Lawton on detailed settlement of Kenney Dam present useful information that does not require further comment by the writer. The paper, its discussions, and the paper "Rockfill Dams and Construction Problems," by D. J. Bleifuss and James P. Hawke (ASCE Proc. Paper 514) give full coverage of Kenney Dam.

A most valuable and comprehensive paper on geology, design, construction and performance of Cheakamus Dam was recently presented by Karl Terzaghi, Hon. M. ASCE.[2] Cheakamus Dam was completed at the end of 1957. Its performance in terms of settlement and leakage during its first 2 yr of operation is excellent. As reported in Mr. Terzaghi's paper,

> "The settlement of the crest of the dam is a maximum above the old river channel where the height of the embankment is greatest (90 ft). At that point, the settlement of the crest increased during the first two years after completion of the dam at a decreasing rate from zero to 0.12 ft and the ultimate settlement will hardly exceed 0.25 ft . . . . The total loss of water due to leakage at full reservoir is about 0.7 cfs which is inconsequential. Therefore, it was justified to make the bulk of the impervious section of the dam out of relatively inexpensive, unprocessed Rubble Creek Wash. All of the water coming out of the contact layer beneath the rockfill section of the dam enters an inverted filter layer and the discharge of the springs coming out of the east slope of the river channel downstream from the dam is practically independent of the elevation of the water level in the reservoir. Therefore, there is no danger of a failure of the dam by piping."

---

[a] June, 1958, by William G. Huber.

[1] Cons. Engr., Vancouver, B. C.

[2] "Storage Dam Founded on Landslide Debris," by Karl Terzaghi, Journal, Boston Soc. of Civ. Engrs., January, 1960.

KASASKAI CENTRAL CORE DAM, AFGHANISTAN[a]

Closure by G. F. Sudman

G. F. SUDMAN,[1] M. ASCE.—Mr. Lawton mentions the desirability of comparing the relative productivity versus remuneration of Afghan labor as compared to American labor. Unfortunately the records which might produce any factual information are not available. Many factors would enter into such a comparison such as costs of food and housing, costs of training native personnel, and the somewhat inefficient use of cheap labor that is often trained to perform a single function. It was found, however, that properly trained Afghan personnel, paid only a fraction of what an American would receive, often proved to be practically as efficient as Americans in performing the particular operations for which they were qualified.

[a] August, 1958, by G. F. Sudman.
[1] Formerly Chf. Design Engr., Internatl. Engrg. Co., San Francisco, Calif.

ROCKFILL DAMS: REVIEW AND STATISTICS[a]

Addendum to Closure

ADDENDUM TO CLOSURE.—The following data, pertaining to the rockfill dams in the Union of South Africa, were received from the Director of Water Affairs, Pretoria:

Name of dam	Type and height, in feet	Name of river	Year of completion	Slopes	
				Upstream face	Downstream face
BEAUFORT WEST	Cmr 185	Gamka	1955	1.25	1.40
KALKFONTEIN	Cmr 113	Riet	1938	1.20	1.40
MOSSEL RIVER	Cmr 81	Mossel	1948	1.40	1.40
OHRIGSTAD	Cmr 170	Ohrigstad	1955	1.40	1.40
PRINS RIVER	Cmr 113	Prins	1913	1.25	1.40
RUST DER WINTER	Cmr 110	Elands	1934	1.20	1.40

[a] August, 1958, by F. W. Scheidenhelm, John B. Snethlage, and Arthur N. Vanderlip.

ROCKFILL DAMS: THE DERBENDI KHAN DAM[a]

Closure by Calvin V. Davis

CALVIN V. DAVIS,[1] F. ASCE.—Thanks are expressed to Mr. Sakurai for his discussion of this subject. The data concerning the Makio Dam is a significant addition. It was of interest to note that the steep valley cross section and relatively low shear strength of the core material, common to both the Makio and Derbendi Khan dams, dictated the use of a thin vertical core section in both cases rather than a sloping core section.

Because of the similarity between the two projects and considering that construction of the Makio Dam is in an early phase, it is believed to be of interest to discuss the most recent construction experiences at the Derbendi Khan Dam. River closure and diversion through the 6 and 9 m tunnels were made in June, 1958. Since that date the wet season cofferdam has been completed and a total of 2,000,000 cu yd of fill have been placed. Fill operations have lagged behind schedule because of delay in completion of the grout curtain along the center of the cutoff trench. This problem has been remedied by constructing a concrete gallery in the rock along the ungrouted portion of the foundation. Curtain grouting is being completed from the gallery while placing core material over the gallery. Final cleanup of the excavated foundation rock is being made just ahead of fill placement to minimize the length of time which the compaction shales are exposed to desiccation by weathering. This procedure has worked out quite well and will be continued throughout construction.

Although design assumptions have been confirmed in general as construction has proceeded, foundation conditions were discovered to be somewhat more critical than originally determined from the exploration program. Initial grouting operations, including extensive water pressure testing of the grout and check holes, disclosed that the top 10 m of rock were quite leaky. Because of these conditions, the grouting program has been expanded to include consolidation grouting along the dam axis on either side of the main grout curtain and within minor fault zones. Consolidation grouting consists of six rows of holes on either side of the main line of curtain grout holes spaced 1.6 m on centers both ways.

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[a] August, 1958, by Calvin V. Davis.

[1] Pres., Harza Engrg. Co., Chicago, Ill.

ROCKFILL DAMS: NANTAHALA SLOPING CORE DAM[a]

Closure by James P. Growdon

JAMES P. GROWDON,[1] F. ASCE.—In his discussion, Mr. Bowman points out, with great clarity, the essential differences between the sloping earth core rockfill dams and other types. The suggestions he offers are most flattering to the writer.

[a] August, 1958, by James P. Growdon.
[1] Cons., Aluminum Co. of America, Pittsburgh, Pa.

ROCKFILL DAMS: DAMS WITH SLOPING EARTH CORES[a]

Closure by James P. Growdon

JAMES P. GROWDON,[1] F. ASCE.—It has been properly pointed out by Mr. Mundal that certain features of a rockfill dam require considerable care during construction. His comments are worth considerable emphasis.

[a] August, 1958, by James P. Growdon.
[1] Cons., Aluminum Co. of America, Pittsburgh, Pa.

ROCKFILL DAMS: THE PERFORMANCE OF SEVEN SLOPING CORE DAMS[a]

Closure by James P. Growdon

JAMES P. GROWDON,[1] F. ASCE.—It is a pleasure to report, in reply to Mr. Lawton, that there has been no lateral movement of the fill on the abutments toward the maximum section on any of the dams referred to by the writer.

Fuse plugs for the control of seldom-used spillways should have a pilot channel which will wash out when overtopped by 1 ft (more or less) of water. The rest of the fuse plug should never be overtopped at all; it should wash out laterally at a rate predetermined by the designer and constructed of materials which will meet the designer's specifications.

[a] August, 1958, by James P. Growdon.

[1] Cons., Aluminum Co. of America, Pittsburgh, Pa.

ROCKFILL DAMS: WISHON AND COURTRIGHT CONCRETE FACE DAMS[a]

Closure by J. Barry Cooke

J. BARRY COOKE,[1] F. ASCE.—Mr. G. I. Davey has presented data on two economical gunite face rockfill dams. Mr. Terzaghi has given a rational discussion of the functions of the cutoff of a concrete face rockfill dam and has suggested further economy. The paper and the two discussions emphasize the particular importance and possibility of practical and economic construction in this type of dam.

The recently constructed 75-ft high Corella River Dam and the 87-ft high Leichard River Dam, described by Mr. Davey, are particularly economical. The 1.25 to 1 (horizontal to vertical) slopes, 2 ft or 2 ½ ft thick placed rock and thin gunite membranes are practicable and certainly very safe for this height of dam. The Upper Bear River Dam and Meadow Lake Dam, described[2] in this symposium, with steeper slopes (0.5 to 1 and 0.75 to 1), and less competent rockfill are both in good health as they approach 60 yr of age. Though the very steep slopes are neither economical nor recommended today, the experience testifies to the substantial safety of the more conservatively sloped dams of today's design and construction standards.

The sluicing by 1-in. nozzles plus watering the loads in the truck beds, though suitable for the 30-ft lifts and for the low dams, would not be considered by the author to be suitable for higher lifts on higher dams. The excavation of pockets of spalls that accumulate at the top of high lifts is considered to require jets of 2-1/2 to 3 in. in diameter. It is interesting to note that with care in dumping, the 1.25 to 1 slope was maintained in the 20-ft to 30-ft high lifts. Courtright Dam slopes were equally steep where the low topping lifts were used in the top 40 ft of the dam. The slopes at 60 ft stations were, in relation to 1 vertical: 1.25, 1.27, 1.25, 1.15, 1.15, 1.11, 1.19, 1.23, 1.12, 1.21, and 1.19. On the slope 40 to 80 ft below the crest in elevation, the slopes were: 1.37, 1.36, 1.37, 1.17, 1.27, 1.20, 1.34, 1.40, and 1.25.

Mr. Terzaghi has carefully reviewed the functioning of the cutoff for the impervious face rockfill type of dam. His conclusions, with which the author is in agreement, are: 1) the function of the cutoff is to reduce seepage or leakage loss and does not involve safety of this type of dam; 2) cutoff trench excavation is very expensive and, even when done with care, can fracture and loosen adjacent rock; 3) grouting provides an effective and economical cutoff curtain; and 4) a suitable and economical cutoff can be obtained by grouting under a rock-anchored slab.

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[a] August, 1958, by J. Barry Cooke.

[1] Super. Civ. Engr., Pacific Gas and Elec. Co., San Francisco, Calif.

[2] "Rockfill Dams: Salt Springs and Lower Bear River Concrete Face Dams," by I. C. Steele and J. B. Cooke, Proceedings, ASCE, Vol. 84, No. PO 4, August, 1958.

Mr. Terzaghi mentions a transition layer downstream from the cutoff which would prevent foundation rock from rising. The placed rock with chinked voids serves this purpose at the contact of placed rock and foundation. Conditions are such that the surface foundation rock downstream from the cutoff should not rise to increase seepage and could not rise to cause a blowout under the cutoff. To a substantial distance downstream from the axis the downward pressure of the self-draining rockfill on the foundation exceeds 100% uplift.

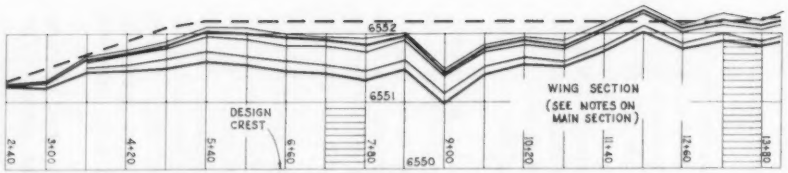
The trend to shallower cutoff walls for Pacific Gas and Electric Company dams has been due to the high cost and the feeling that depth does little to decrease seepage and improve safety. To excavate a narrow trench requires drill holes at 9-in. to 12-in. spacing, about 40 ft of drill hole per cubic yard of excavation and expensive handling of muck. In jointed rock the loosening of the rock in the walls requires going to greater depth to meet a design requirement of a specified depth with competent walls. An excavated and concrete backfilled cutoff is very expensive compared to a grout curtain.

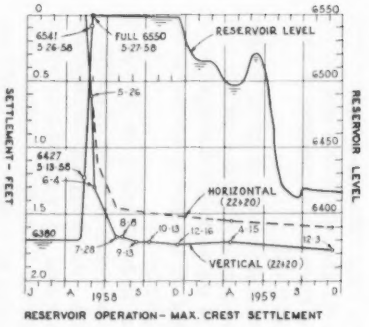
Mr. Terzaghi emphasizes the effectiveness of grouting. In the cutoffs at the company's recent dams, grout pipes have been angled in the cutoff pour to make the drill holes cross specific seams or bedding planes in addition to pattern grouting. A grout curtain under a rock-anchored slab should provide a safe, effective and economical cutoff.

WISHON DAM

Vertical crest settlement for the first 2 yr after completion of the rockfill is shown in detail on Fig. 22. The reservoir operation is also shown. The dashed line shows the crest profile as designed and the line dated 12-13-57 (December 13, 1957) shows the position of the crest on the day dumped rockfill was completed. The actual crest was generally at only one or two-tenths of a foot lower than design elevation upon completion of the rockfill. Between December 13, 1957 and April 13, 1958, while the fill aged for the first 4 months with, essentially, an empty reservoir, the movement was small (only 0.10 ft at the 150-ft high Station 7+80 and 0.32 ft at the 249-ft high Station 21+60). The major portion of settlement took place between 5-13-58 (May 13) and 6-4-58 (June 4), which period included the 13 days of rapid filling and the first 9 days of full reservoir. The next reading was on 7-28-58 (July 28), 54 days later and about 2 months after the reservoir was full. The 7-28-58 reading is the last one plotted in the form of a profile because the settlement has been so small since that time. The tabulation shows six readings between 7-28-58 and 12-3-59 (December 3), which record very small movements during 1 yr and 4 months when the reservoir operation was as shown on Fig. 22. Between 12-16-58 (Dec. 16) and 4-15-59 (April 15), while the reservoir level went down, the crest "raised" 0.01 ft to 0.02 ft and at only 4 of the 51 crest settlement points did the points go downward slightly.

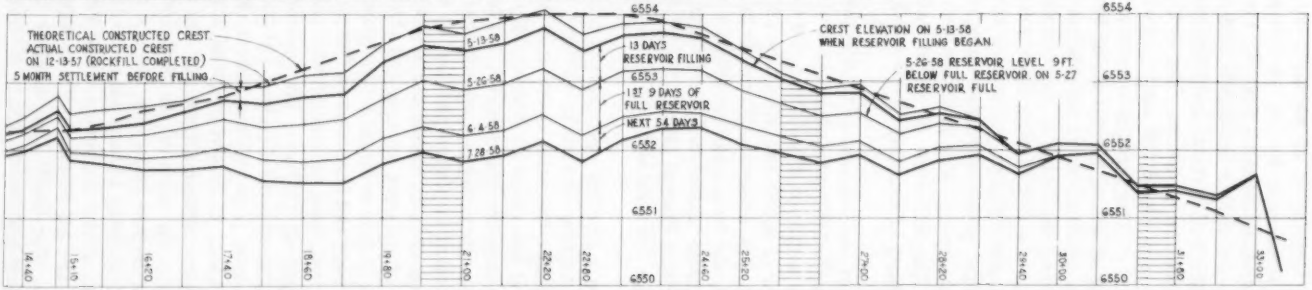
The tabulation on Fig. 22, in addition to all of the crest readings of vertical settlement, shows the height at each section and the settlement, in percentage of height, for each section. The percentage of height figure is based on the 12-3-59 (Dec. 3) reading which is 1 yr and 5 months after the reservoir was first filled. The settlement at the maximum section of the wing section, Station 7+80, is 0.39% of the height or 0.58 ft. The settlement at the maximum section of the main section, Station 22+20 is 0.69% of the height, or 1.77 ft. This is consistent with measurements of settlement on other well constructed dumped





RESERVOIR OPERATION - MAX. CREST SETTLEMENT

HEIGHT AT AXIS - FT		45	70	83	108	123	133	130	128	138	150	138	136	116	110	106	98	98	98	98	98
12-13-57 TO 4-13-58		01	01	06	04	06	—	06	—	04	10	04	04	—	06	06	—	—	05	10	08
5-13-58	13 DAYS	00	00	00	00	00	00	00	00	00	00	00	00	00	00	00	00	00	00	00	00
5-26-58	9 "	00	05	02	02	04	08	09	14	09	11	09	06	06	06	05	06	05	05	05	07
6-4-58	54 "	00	05	11	17	22	29	34	36	37	40	36	32	28	26	24	24	23	21	20	21
7-28-58	12 "	00	06	15	23	30	39	45	49	50	52	50	45	40	36	30	33	33	30	28	28
8-9-58	35 "	01	07	16	24	31	41	47	50	51	53	51	46	40	37	34	34	32	30	28	28
9-13-58	30 "	02	08	17	25	32	41	47	50	52	54	52	47	41	37	35	35	34	31	29	31
10-13-58	64 "	00	08	17	25	33	43	47	51	53	54	51	46	40	37	34	33	32	30	30	32
12-16-58	121 "	03	09	18	27	34	43	49	52	53	56	53	48	42	39	36	35	37	32	30	31
4-15-59	232 DAYS	02	08	18	25	32	41	48	51	53	55	52	47	41	37	35	35	34	32	30	31
12-3-59		03	10	20	28	35	45	51	54	55	58	54	50	45	40	38	37	35	33	31	33
% OF HEIGHT		0.07	0.14	0.24	0.26	0.29	0.34	0.39	0.42	0.40	0.39	0.39	0.37	0.39	0.36	0.36	0.38	0.36	0.33	0.32	0.34



103	103	110	130	155	166	176	183	176	168	214	264	254	249	256	244	245	230	224	208	212	200	170	130	134	125	103	70	60	56	47	HEIGHT AT AXIS - FT	
17	—	24	27	25	17	21	26	31	30	27	27	24	32	24	22	17	14	17	15	09	06	10	07	09	—	—	—	—	—	—	12-13-57 TO 4-13-58	
00	00	00	00	00	00	00	00	00	00	00	00	00	00	00	00	00	00	00	00	00	00	00	00	00	00	00	00	00	00	00	13 DAYS	5-13-58
07	10	10	14	18	25	28	34	40	37	54	55	60	61	62	59	55	52	47	43	36	35	30	22	15	10	03	01	02	00	00	9 "	5-26-58
24	28	31	40	52	65	72	83	96	96	113	121	128	130	131	127	121	114	111	97	85	81	72	63	50	37	20	11	10	06	06	54 "	6-4-58
32	38	41	53	69	86	97	112	125	130	148	159	166	167	168	163	154	142	132	123	109	103	92	81	67	50	30	17	12	09	05	12 "	7-28-58
33	39	42	54	69	87	98	112	127	131	149	159	166	168	169	163	155	146	134	124	111	105	93	82	68	52	32	18	12	09	05	35 "	8-9-58
34	41	44	55	70	88	99	113	128	131	150	158	167	171	172	166	157		136	126	114	107	98	84	70	54	33	20	15	08	07	30 "	9-13-58
33	40	43	54	70	87	99	113	128	131	149	161	168	171	172	166	157	148	135	126	115	107	95	84	70	53	33	20	16	11	06	64 "	10-13-58
35	41	45	56	72	90	102	116	131	135	152	163	170	173	175	168	157	148	136	127	113	107	94	85	70	53	33	19	14	10	06	121 "	12-16-58
34	41	44	56	72	90	102	115	130	134	152	162	168	171	172	167	158	149	136	126	115	107	96	85	70	53	32	19	14	11	06	232 DAYS	4-15-59
36	43	47	59	75	94	106	120	135	139	156	167	173	176	177	172	163	153	140	131	118	112	99	88	73	56	35	20	16	10	06	12-3-59	
036	042	043	045	049	057	060	066	077	083	073	063	068	071	069	071	067	067	063	063	056	056	058	067	055	045	034	029	023	018	013	% OF HEIGHT	

FIG. 22.—VERTICAL CREST SETTLEMENT—WISHON DAM

rockfill dams. The settlement of sections other than these two highest and central sections is influenced by the steepness of the abutments and by the foundation contours and, as seen on Fig. 22, the settlement expressed in percentage of height at axis varies between 0.07% and 0.83%.

Fig. 23 shows profiles of vertical and horizontal components of crest settlement along with foundation profiles. The settlement is plotted relative to a straight line datum which shows the smoothness of the settlement compared to the irregular foundation. The irregular lines of Fig. 22 are due to the distorted scale and the plotting of actual elevations of the points.

It is of interest to see that the horizontal movements of points on the 150-ft high wing section exceed the vertical movements, whereas it is the opposite in the central portion of the 250-ft high main section. As is typical of concrete face dam movement, the ratio of horizontal component to vertical component of settlement of a crest point increases as points are taken for sections from the center to each abutment. For example, the ratio is 0.86 at Sta. 21+00 and 2.00 at Sta. 30+00. The influence on crest settlement of the very irregular foundation in the vicinity of Stations 18+00 and 21+00 is seen to be rather small and not abrupt.

Fig. 24 gives all of the vertical and horizontal movements, during the first year, of points above the 1959 minimum reservoir level of 6412 ft. Also shown on Fig. 24 are cross sections illustrating the settlement. The vertical movements at the corners of slabs are shown to indicate the moderate offsets between adjacent slabs. The offsets are seen to be 0.01 ft to 0.02 ft. At maximum height section 22+20, the first year movement normal to the face was 2.8 ft at elevation 6440 ft, a little above the mid-height of the dam. At Station 19+80, the movement is greater than at Station 18+60 due to the lower foundation profile downstream from the axis and due to 19+80 being closer to the center of the main section.

Table 1 and Table 2 give the opening and closing of vertical and horizontal joints for the wing section and main section, respectively. The movements were very small. Where the joint opening is smaller than noted in the paper, it is because the joint has since closed slightly. All joint movements are seen to be small and it is possible that the very large placed rock did not permit the joints to act as much as they might have and that the underlying fill had greater components of movement in planes parallel to the face.

Table 1 for the wing section shows slight (0.01 ft) opening of the crest A-line joints near the ends of the wing section and slight closing (0.01 ft) of the joints near the center. The B-line vertical joints generally opened 0.01 to 0.02 ft before filling the reservoir and then closed during the first years cycle of reservoir operation. The resultant movement was only 0.01 ft opening or closing as shown on the tabulation as TOTAL. The opening between 9-1-57 (Sept. 1) and the beginning of filling on 5-13-58 (May 13) was probably due to shrinkage of the slabs. The horizontal joint, B-line, was constructed with a 3/4 in. redwood board and Z copper seal. It closed by from 0.01 ft to 0.02 ft.

Table 2 for the main section shows, for the crest A-line, the usual opening of vertical joints near the ends of the dam and closing near the center. The vertical joints at the B-line generally did not move or opened slightly before filling began on 5-13-58 (May 13) and all closed (except at Station 19+20) during first years operation of the reservoir. The resultant movements of vertical joints at the B-line are all closing with greatest closures in the central part of the main section. For the C-line similar comments as for B-line apply. The B-line and C-line horizontal joints all closed moderately, 0.01 ft to 0.02

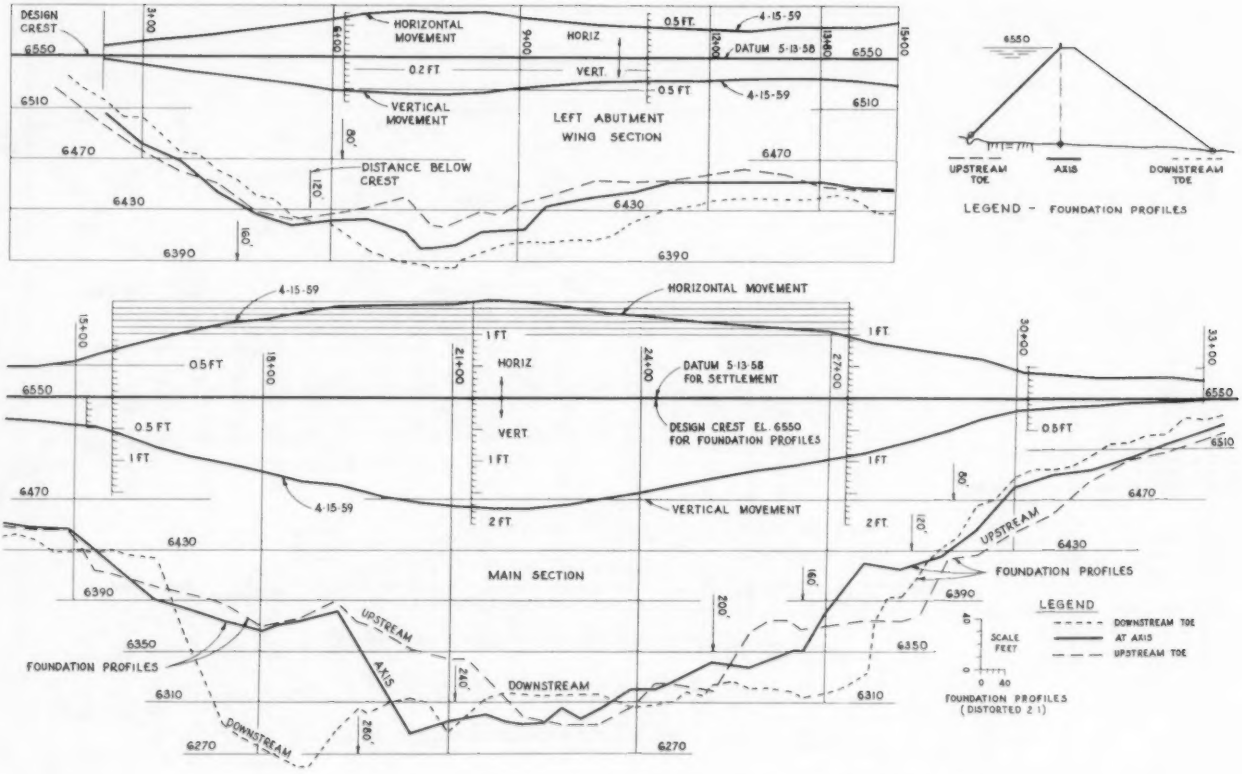


FIG. 23.—PROFILES OF HORIZONTAL AND VERTICAL CREST SETTLEMENT-FOUNDATION PROFILES—WISHON DAM

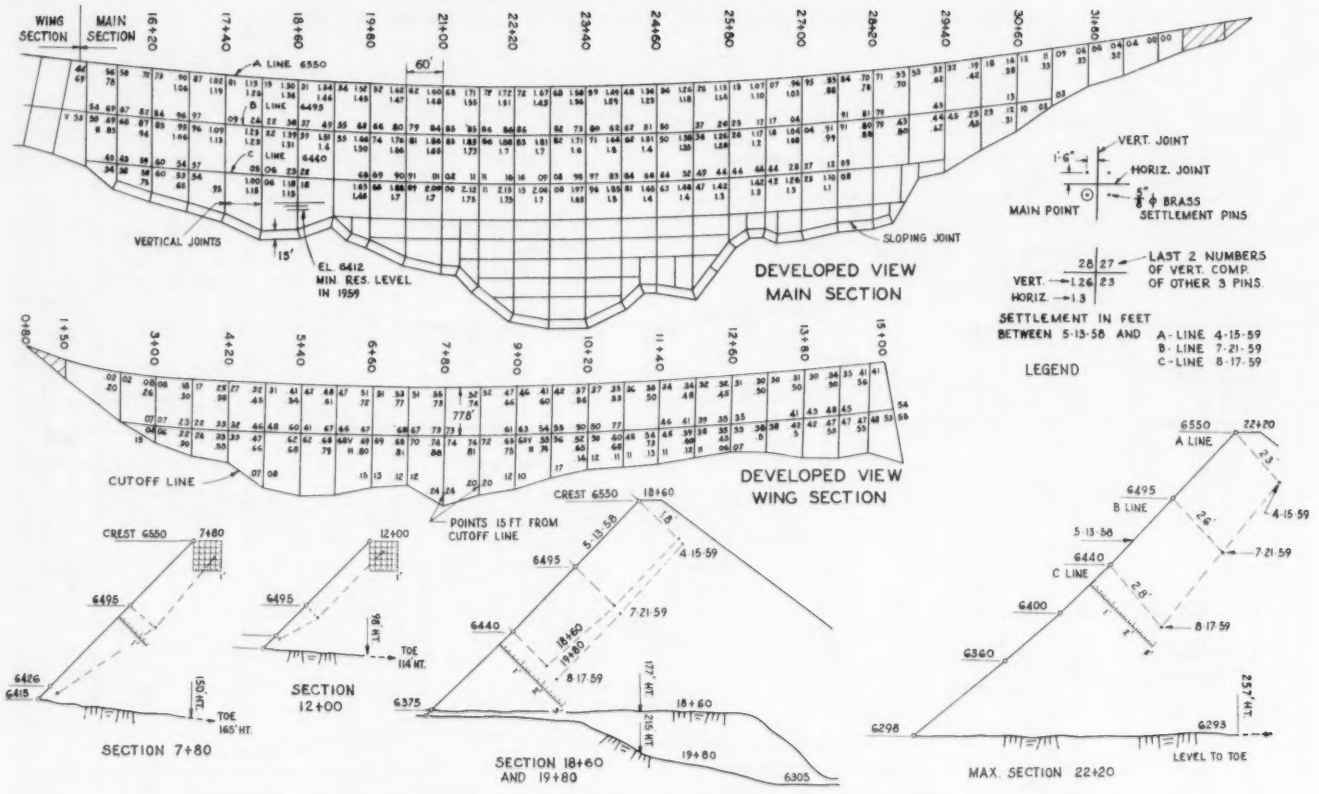


FIG. 24.—FIRST YEAR VERTICAL AND HORIZONTAL SETTLEMENT DATA—WISHON DAM

TABLE 1.—VERTICAL AND HORIZONTAL JOINT MOVEMENTS—WING SECTION—WISHON DAM

	VERTICAL JOINTS - FT (+) = Opening and (-) = Closing				HORIZONTAL JOINTS - FT (+) = Opening and (-) = Closing		
	A-LINE	B-LINE			B-LINE		
	5-13-58 To 12-3-59	9-1-57 To 5-13-58	5-13-58 To 7-21-59	TOTAL	10-30-57 To 5-13-58	5-13-58 To 7-21-59	TOTAL
2+40	+ .01						
3+00	0		- .01	- .01	0	0	0
3+60	+ .01	+ .02	0	+ .02	- .01	- .01	- .02
4+20	0	0	- .01	- .01	- .01	0	- .01
4+80	0	- .01	0	- .01	- .01	0	- .01
5+40	- .01	0	0	0	0	- .01	- .01
6+00	- .01	- .01	0	- .01	- .02	0	- .02
6+60	0	+ .01	- .02	- .01	0	- .01	- .01
7+20	- .01	- .01	0	- .01	- .01	0	- .01
7+80	- .01	0	0	0	- .01	0	- .01
8+40	- .01	0	0	0	0	- .01	- .01
9+00	- .01	+ .02	- .02	0	0	- .01	- .01
9+60	- .01	+ .02	- .03	- .01	0	- .02	- .02
10+20	0	+ .02	- .02	0	- .01	- .01	- .02
10+80	0	0	0	0		- .02	
11+40	0	+ .02	- .02	0		- .02	
12+00	0	0	0	0	- .01	- .01	- .02
12+60	0	+ .01	0	+ .01	0	- .02	- .02
13+20	+ .01	0	0	0		- .01	- .01
13+80	0	+ .02	- .01	+ .01	- .01	0	- .01
14+40	0	+ .02	- .01	+ .01		- .01	
14+90	+ .03	0	0	0		- .01	

TABLE 2.—VERTICAL AND HORIZONTAL JOINT MOVEMENTS—MAIN SECTION—WISHON DAM

	VERTICAL JOINTS - FT (+) = Opening and (-) = Closing							HORIZONTAL JOINTS - FT (+) = Opening and (-) = Closing			
	A-LINE	B-LINE			C-LINE			B-LINE	C-LINE		
	5-13-58 To 12-3-59	10-30-57 To 5-13-58	5-13-58 To 7-21-59	TOTAL	10-30-57 To 5-13-58	5-13-58 To 8-17-59	TOTAL	5-13-58 To 7-21-59	10-30-57 To 5-13-58	5-13-58 To 8-17-59	TOTAL
15+10	+ .02		- .03	- .03				- .01			
15+60	+ .01	- .01	- .01	- .02	- .01	- .01	- .02	- .01		- .02	
16+20	0	0	- .01	- .01	0	- .01	- .01	0	- .01	- .01	- .02
16+80	0		0	0	- .02	- .01	- .03	- .01	- .01	- .02	- .03
17+40	+ .01				+ .01	- .02	- .01		+ .01	- .01	0
18+00	0		0	0	- .01	- .01	- .02	0	0	- .02	- .02
18+60	0	0	- .01	- .01	0	- .01	- .01	- .02	+ .01	- .02	- .01
19+20	0	- .02	+ .02	0	+ .01			- .02	0		
19+80		0	- .03	- .03	0	- .02	- .02	- .01	+ .01	- .03	- .02
20+40	- .01	0	- .01	- .01	+ .01	- .02	- .01	- .01	+ .01	- .02	- .01
21+00	- .02	0	- .02	- .02	0	- .02	- .02	- .01	0	- .02	- .02
21+60	- .01	+ .01	- .03	- .02	- .01	- .02	- .03	- .01	+ .01	- .01	0
22+20	- .03	- .01	- .03	- .04	0	- .02	- .02	0	+ .01	- .01	0
22+80	- .02	+ .01	- .01	0	0	- .02	- .02	- .01	0	- .01	- .01
23+40	- .01	- .01	- .02	- .03	+ .01	- .02	- .01	- .01	0	- .01	- .01
24+00	- .02	+ .01	- .03	- .02	0	- .01	- .01	- .02	+ .01	- .01	0
24+60	- .01	0	- .02	- .02	- .01	- .01	- .02	- .02	0	- .01	- .01
25+20	0	+ .01	- .02	- .01	0	- .02	- .02	- .02	0	- .02	- .02
25+80	0	+ .01	- .02	- .01	0	- .01	- .01	- .01	+ .01	- .01	0
26+40	0	0	- .02	- .02	- .01	- .01	- .02	- .02	0	- .03	- .03
27+00	0	+ .01	- .02	- .01	0	- .01	- .01	- .01	0	- .02	- .02
27+60	- .01	+ .02	- .02	0	- .01	- .01	- .02				
28+20	0	0	- .01	- .01	0	- .01	- .01	- .01			
28+80	0	0	- .01	- .01		- .01		- .02			
29+40	+ .02	+ .01	- .02	- .01				- .02	- .01	+ .01	0
30+00	+ .06	0	0	0							
30+60	+ .01	+ .01	0	0							
31+20	+ .01										
31+80	0										
32+40	+ .01										
33+00	+ .01										
33+40	+ .01										

ft with two 0.03 ft closures on the C-line. These closures are 1/6 to 1/2 of the 0.06 ft thick redwood board. The corner measurements on Fig. 24 show that the upper slab moves down toward the lower slab.

Leakage from the left abutment to Station 12+00 of the wing section is isolated from that from the rest of the dam. It reached a peak of 3.2 cfs for full reservoir and has steadily decreased. It was 0.9 cfs for reservoir level of 6520 ft elevation, the maximum level reached in 1959. In October, 1959, when the face of the wing section was exposed, cracks of about 1/32 in., paralleling portions of the abutment, were observed. They were chipped 1 in. deep and primed and filled with Igas mastic to prevent deterioration due to frost action more than to reduce seepage.

Upon initial filling, leakage from the main section was excessive. The source of leakage was located and underwater repairs were made to prevent loss of water. Some of the sloping joints had failed, possible due to a construction cause. The details of what actually occurred will be known when the Wishon Dam is unwatered through normal operation in a future year.

COURTRIGHT DAM

Courtright Dam was completed in October, 1958, several months after the paper was written. In 1959, the runoff was subnormal and the reservoir filled to only two-thirds the height of the dam. The available settlement data at this time is therefore limited. Fig. 25 presents a study of construction settlement and the movement of the dam due to the partial filling in 1959.

As shown on the elevation and section of dam on Fig. 25, the top 80 ft of rockfill was dumped between 7-15-58 (July 15) and 9-15-58 (Sept. 15). The two profiles of C-line vertical settlement and the maximum section showing settlement show the movements that took place during that period of construction, and also some movement before and after that period.

Vertical settlement of C-line during construction (temporary points) of Fig. 25 gives the movement of temporary points on the placed rock behind the slabs at C-line elevation. The readings are taken on different days than the readings on permanent points. The temporary point profile is shown only to confirm the permanent point readings that show such rapid movements during construction. The movements will be discussed with reference to the permanent points.

Vertical settlement of C-line (permanent points) of Fig. 25 gives the movement of each C-line point from the day the slab below C-line was poured to 11-20-59 (Nov. 20), about 1 yr and 4 months later. 3 ft of vertical settlement took place where the C-line was 220 ft above foundation during dumping of 80 ft of rockfill above the C-line as shown on Fig. 25. This is about 1.4%. The reason the movement, as measured, is greater at Station 4+80, than at the higher sections of 5+40 or 6+00, is that a small fill as shown above C-line was dumped before the measurements that are shown were started. In the year and one month between the 10-7-58 (Oct. 7) reading and the 11-20-59 (Nov. 20) reading, the maximum C-line movement was only 0.2 ft due to aging of the fill and the water rising 200 ft up on the 300-ft high dam and remaining there. This is significantly low compared to the 3-ft movement during construction. Of course, a substantial future movement will take place when the reservoir is first completely filled.

Maximum section, station 5+40,—showing settlement of Fig. 25 gives the settlement from the day each slab was poured until the various dates shown.

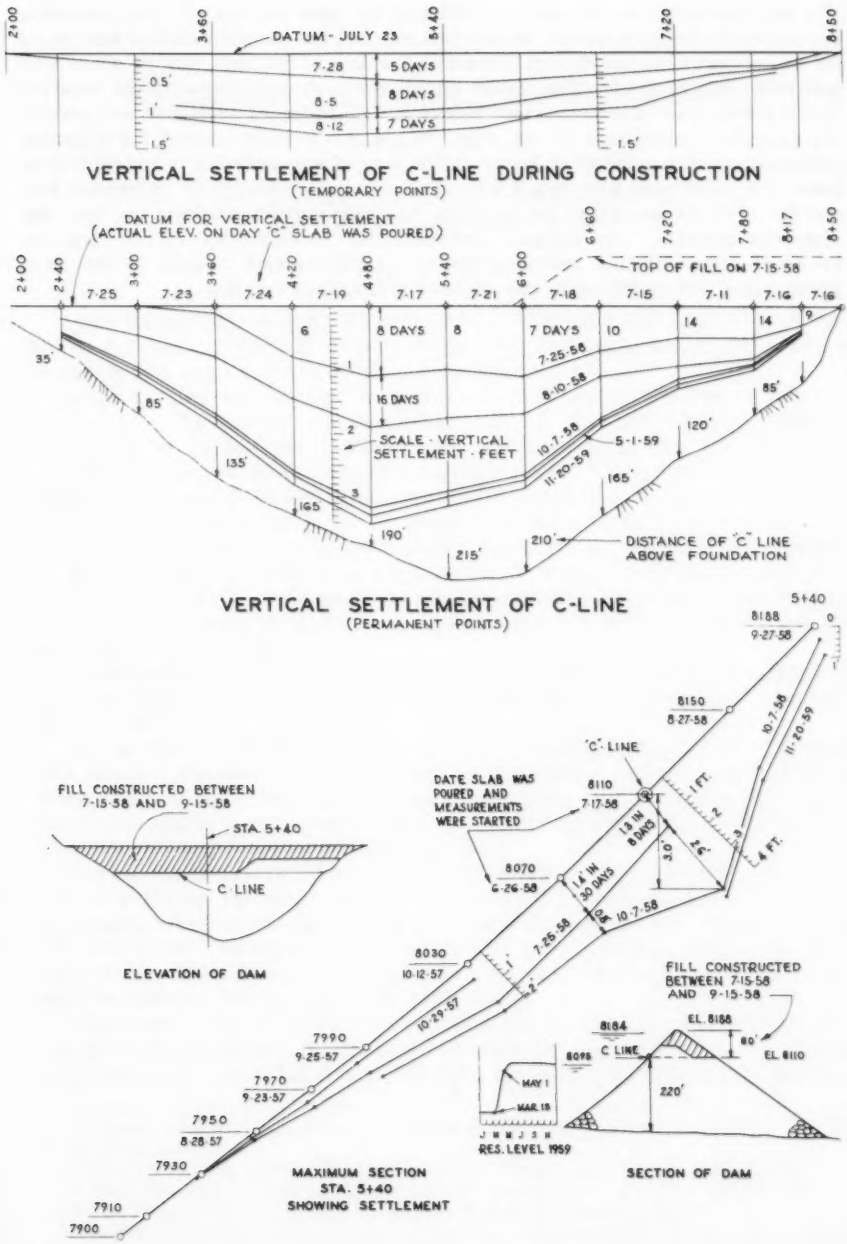


FIG. 25.—SETTLEMENT DATA—COURTRIGHT DAM

The reference line is the design location of the face and each row of slabs was poured to the design position. As the dam settled, the placed rock was steepened so as to return to the design lines and the top of the next row of slabs was poured to design grade. The bottom of the new slab was poured to the location of the lower slab. The settlement points are in the top of each slab and record the complete movement of the slab. While the C-point moved 2.6 ft during construction, the point 40 ft lower on the face moved only 0.8 ft and 40 ft still lower the movement was only 0.4 ft. The distance between the reference line and the 10-7-58 line gives the total construction settlement from the time the slab was poured. The distance between the 10-7-58 (Oct. 7) line and the 11-20-59 (Nov. 20) line indicates the small settlements caused by the first years aging and by filling to two-thirds the height of the dam.

ROCKFILL DAMS: THE PARADELA CONCRETE FACE DAM[a]

Closure by Luis Henrique Gomes Fernandes, Edgard de Oliveira, and Nuno de Vasconcelos Porto

LUIS HENRIQUE GOMES FERNANDES,[1] EDGARD DE OLIVEIRA,[2] and NUNO DE VASCONCELOS PORTO.[3]—As this article is being written, the Paradela reservoir is being filled for the third time.[4]

In the two previous fillings the maximum elevations of 738.8 m. (2,424.0 ft) and 740.8 m. (2,430.6 ft) were reached on July 12, 1958 and April 15, 1959 respectively. In the past year (1959) the two spillways worked for the first time.

Although the maximum leakages during the first filling did not exceed 382 litres per second (13.5 cfs) it was decided to empty the reservoir completely at the end of 1958 to carry out a general inspection of the concrete face, and advantage was taken of the occasion to get a general survey of the settlements of the dam. At the same time a campaign of repair was carried out on the damages discovered, although it was not as overall as would have been convenient due to the short time available between the end of the emptying and the start of the rainy season.

The work involved in this repair campaign cost 600,000$00 Escudos ($21,000 in United States currency). The total cost of the dam of 275,000,000$00 Escudos ($9,600,000).

During the second filling in 1959 a very marked increase in the leakages was noticed, reaching 3,020 litres per sec (106.6 cfs), and in consequence it was decided to empty the reservoir again completely, to carry out the necessary inspection and eventual repair works. The work executed in that year was done at a cost of 2,000,000$00 Escudos ($70,000).

During the two emptyings, both of which were complete, the position of the bottom outlet on the upstream side of the cofferdam was found to be a great advantage, allowing the flow of the river to pass, and also the advantage was realized of the two small drains not only to allow the flow of the volume of water between the dam and the cofferdam but also the flow of the small leakages beneath the foundations of the cofferdam.

Observations and Work Carried Out Until the End of 1958.—During this year and the first filling, that reached the elevation 738.8 m (2,424.0 ft), the normal maximum level being 740.0 m (2,427.9 ft), the leakages went on rising slowly

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[a] August, 1958, by Luis Henrique Gomes Fernandes, Edgard De Oliveira, and Nuno de Vasconcelos Porto.

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[3] Civ. Engr., Hildro Eléctrica do Cávado, S.A.R.L., Porto, Portugal.

[4] In this filling, the level of the reservoir has been maintained around the maximum since the end of December, 1959 until this date (March 10, 1960) and the leakages are on the order of 550 liters per sec. (19.4 cfs.)

until they reached a maximum of 382 litres per sec (13.5 cfs). The decrease in the leakages according to the level during the emptying brought nothing special to note.

The various deformations of the dam were measured at several different periods and include: absolute displacements, represented in plan and in profile (the latter in vertical planes parallel to the vertical plane that contains joint 15): relative displacements, of points to both sides of the joints, that measure the movements of the same joints.

The data on the plans refer to:

1) The first observation of the point studied and serves as the origin for future measurements.

2) The observation immediately before the time when the point studied was reached by the water in the first filling (1957-58). June, 1958 was taken for the points not reached by the water.

3) The observation immediately following the unwatering of the point on emptying the reservoir in 1958. September, 1958 was taken for the points not reached by the water.

4) The observation immediately following the unwatering of the point on emptying the reservoir in 1959.

As can be seen[5] in Fig. 1, until the dates of the survey before the first filling, there were already important settlements, principally at the level of the horizontal joint at elevation 727.25 m (2,386 ft), which, in the area of this joint, were mainly the result of the dumping of the rockfill in the third phase of construction.

The filling of the reservoir simultaneous with the construction operations makes a detailed interpretation of the settlements difficult. It is noted that the lateral movements towards the center of the dam are rather more accentuated on the right side than on the left.

After the first filling, an increase in the settlements in plan were noticed, as was expected, as much downstream as towards the center of the dam. It is noted that on the left bank the settlements in plan are very close to the normal of the level lines on the upstream face. This has not taken place on the right side where the settlements have deviated notably towards the center.

Movements upstream were noticed at the points of observation placed on the joint between the perimetral slabs P and Q especially in the area between the connecting joints 12 and 20, as a result of the raising of this joint by the force of compression in the plane of the slabs, which force came from the closing movement of the joints in that area and causing a displacement of those slabs, as shown in Fig. 6.

In 1958 the maximum displacement downstream was of the order of 0.926 m (3.04 ft) and the maximum lateral displacements towards the center of the dam and measured normally to the vertical planes of the joints, were of 0.152 m (0.50 ft) and 0.291 m (0.95 ft) respectively on the left side and right side.

Until the date of the observations before the first filling there were also important deformations in profile, as can be seen[5] in Fig. 2, chiefly at the level of the horizontal joints at elevations 727.25 m (2,386.0 ft) and 718.125 m (2,356.0 ft), the greater part of which, as has been stated, can be attributed to the dumping of the rockfill in the 3rd phase, and the smaller part to the hydrostatic pressure already being applied to the lower elevation, because the reservoir was in the process of filling as conditions allowed.

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5 Construction and loading in stages, with some important intervals between measurements on adjacent points, caused discontinutiy in settlement lines of upper part of the dam.

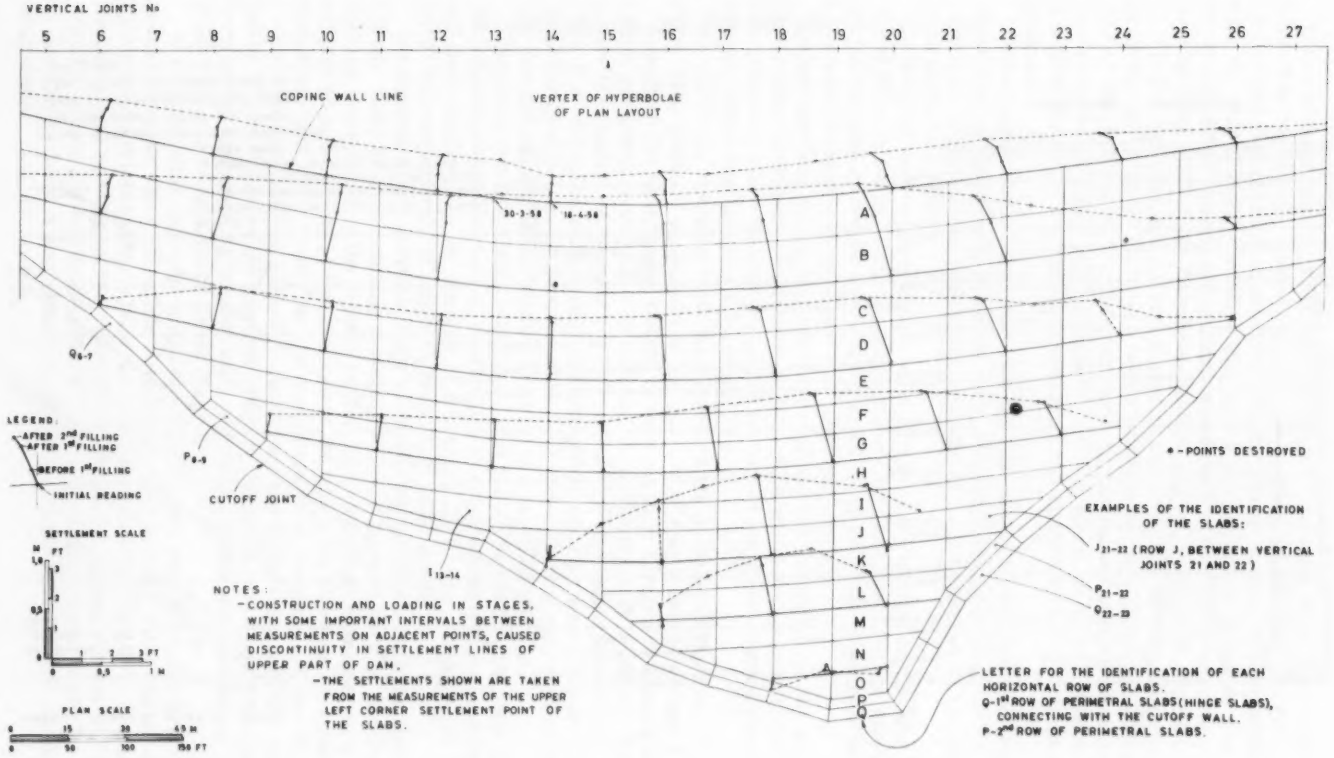


FIG. 1.—PLAN OF THE UPSTREAM CONCRETE FACE SHOWING SETTLEMENT

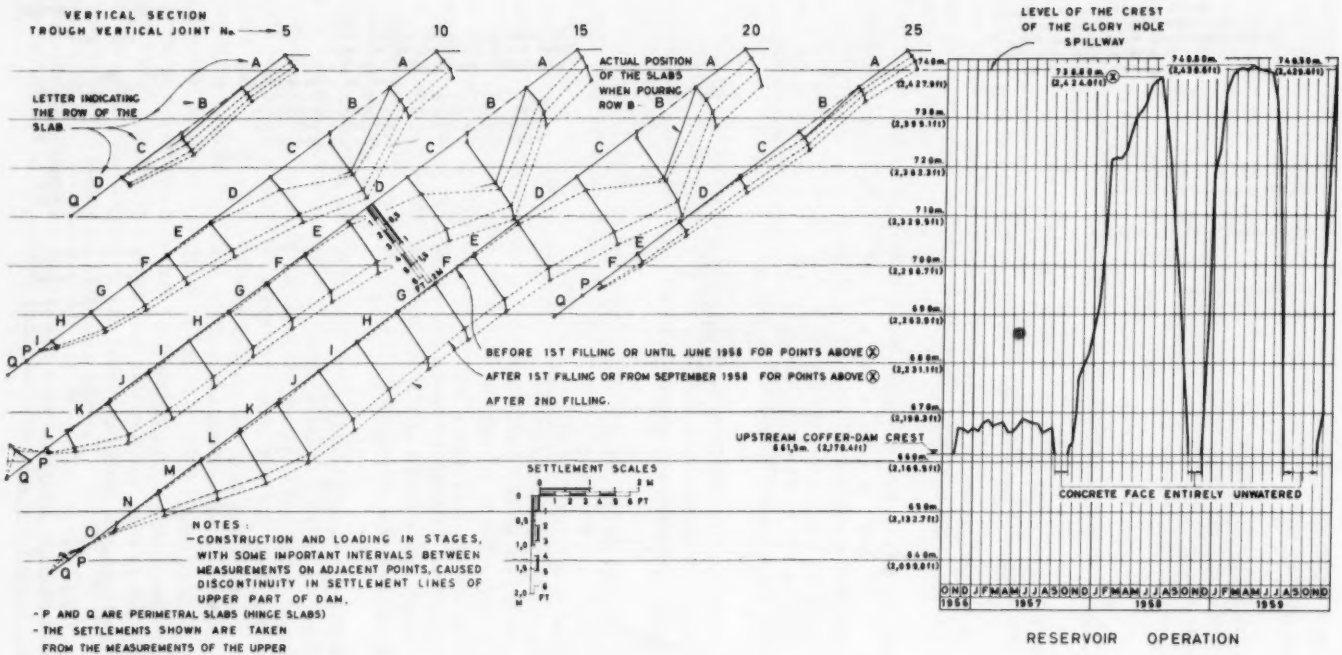

VERTICAL SECTION
TROUGH VERTICAL JOINT No. 5
10
15
20
25
LETTER INDICATING THE ROW OF THE SLAB
ACTUAL POSITION OF THE SLABS WHEN POURING ROW B
BEFORE 1ST FILLING OR UNTIL JUNE 1958 FOR POINTS ABOVE
AFTER 1ST FILLING OR FROM SEPTEMBER 1958 FOR POINTS ABOVE
AFTER 2ND FILLING.
SETTLEMENT SCALES
NOTES :
-CONSTRUCTION AND LOADING IN STAGES, WITH SOME IMPORTANT INTERVALS BETWEEN MEASUREMENTS ON ADJACENT POINTS, CAUSED DISCONTINUITY IN SETTLEMENT LINES OF UPPER PART OF DAM.
- P AND Q ARE PERIMETRAL SLABS (HINGE SLABS)
- THE SETTLEMENTS SHOWN ARE TAKEN FROM THE MEASUREMENTS OF THE UPPER LEFT CORNER SETTLEMENT POINT OF THE SLABS.
LEVEL OF THE CREST OF THE GLORY HOLE SPILLWAY
UPSTREAM COFFER-DAM CREST
CONCRETE FACE ENTIRELY UNWATERED
RESERVOIR OPERATION

FIG. 2.—SECTIONS OF THE DAM SHOWING SETTLEMENT

It is explained that the dam had already settled when the slabs of the rows A and B were poured. That is, the lay out position of the joint between slabs A and B had already settled considerably, for the reasons previously explained, when the placed rock, corresponding to the rows of slabs B and A, was laid. The laying of the placed rock in the area of the B slabs was done so as to connect the joint B-C in its deformed position with the designed position of the joint A-B. The placed rock of the area of the slabs A was then constructed in the designed position. Although some settlements had taken place up to the time of the pouring of the slabs for these two rows, the designed thickness of the concrete was maintained without an attempt being made, at the expense of excessive thickness, to reach the position of the upstream face that was initially designed.

In their general lines these settlements, as well as those resulting from the application of hydrostatic pressure up to elevation 738,80 m (2,424.0 ft), were quite near normal to the profiles of the face in planes parallel to the vertical plane containing joint 15.

In the joint between perimetral slabs (the area between the connecting joints 12 and 20) relative uplifting movements were noticed, detaching the slabs from the placed rock on which they were poured (v.fig.5). The maximum vertical settlement measured to this time was 1.585 m (5.20 ft).

Before the first filling the greater part of the horizontal joints did not show appreciable movement. Only some of the joints of the area constructed in the first phase—to allow the exploitation of the run of the river--showed important closing, with a maximum of 1.6 cm (0.05 ft).

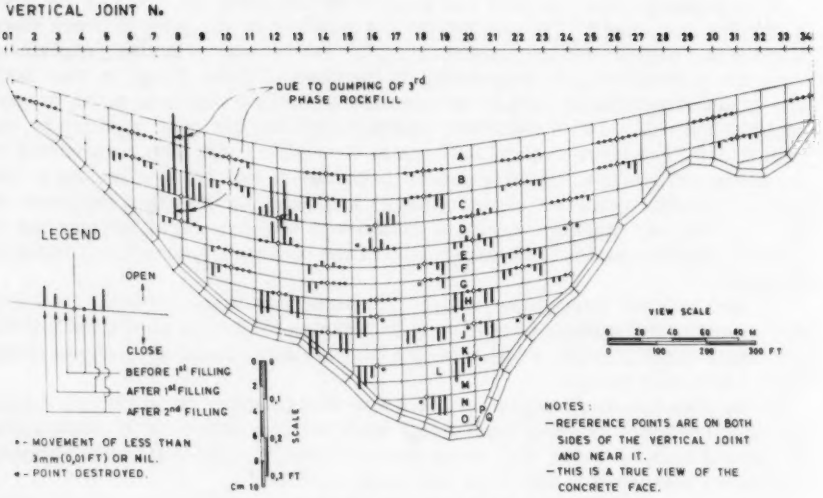
At that time, resulting from the effects of the dumping of the loose rockfill in the third phase of construction, the horizontal joints between slabs A to E, especially in the stretches to the left of vertical joint 16, showed opening movements, particularly in joint C-D between the vertical joints 7 and 13. For example, the maximum opening of 7.0 cm (0.23 ft) was observed in joint C-D, while in D-E (in the same area) the maximum opening was 2.5 cm (0.08 ft). After the first filling the general movement was a closing one, with increases that brought the maximum closing to about 2.3 cm (0.08 ft).

The joints between slabs from A to E, that had opened before the first filling, also showed noticeable closing, which in the cases previously mentioned brought the final positions to an opening of 2.2 cm (0.07 ft) and a closing of 0.2 cm (0.007 ft), respectively at joints C-D and D-E (Fig. 3(a)).

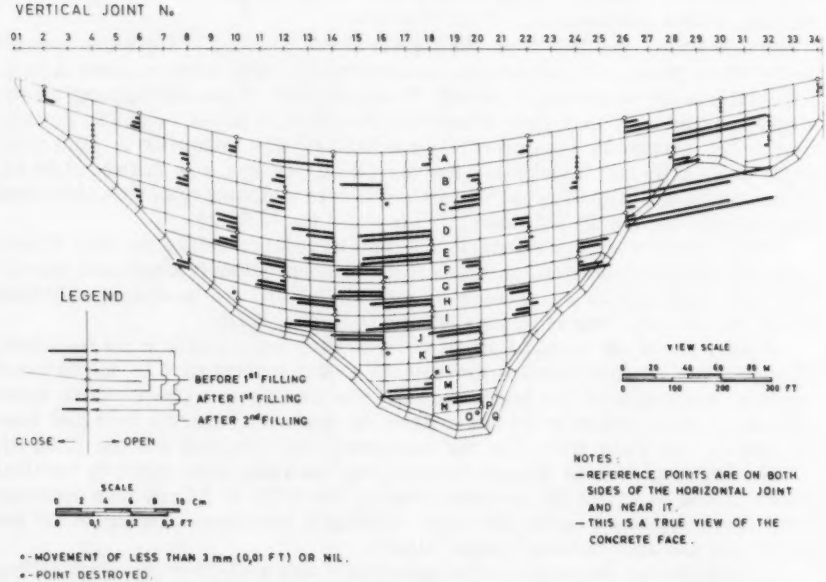
Movements of the vertical joints were already noticed before the first total filling. They involved closing movements in the central part of the dam and opening movements in the lateral areas. The closing movements were more evident at some points in the lower area, in which the concrete face had been completed for some time. At the maximum they reached 2.8 cm (0.09 ft).

The openings showed greater intensity on the right side, between vertical joints 23 and 29, where the greatest were of the order of 2.5 cm. The openings were more moderate on the left side, showing a maximum opening of 1.0 cm (0.03 ft) in the area between joints 0 and 6.

A considerable increase in the movements was seen after the first filling generally in the same direction as had been seen before. Thus, the closing in the central part increased, raising the maximum to 5.6 cm (0.18 ft). In addition, the maximum opening on the right side increased to about 9.8 cm (0.32 ft) at joint 26, and on the left side joints 4 to 6 were slightly closed while the joints to the left of joint 4 had opened slightly to a maximum of 1.0 cm (0.03 ft) (Fig. 3(b)).


VERTICAL JOINT No.
01 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31 32 33 34
DUE TO DUMPING OF 3rd PHASE ROCKFILL
A B C D E F G H I J K L M N O P Q
LEGEND
OPEN
CLOSE
BEFORE 1st FILLING
AFTER 1st FILLING
AFTER 2nd FILLING
SCALE
Cm 10 0,3 FT
VIEW SCALE
0 20 40 60 80 M
0 100 200 300 FT
NOTES :
– REFERENCE POINTS ARE ON BOTH SIDES OF THE VERTICAL JOINT AND NEAR IT.
– THIS IS A TRUE VIEW OF THE CONCRETE FACE.
• – MOVEMENT OF LESS THAN 3 mm (0,01 FT) OR NIL.
• – POINT DESTROYED.

a)–OPENING AND CLOSING OF HORIZONTAL JOINTS


VERTICAL JOINT No.
01 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31 32 33 34
A B C D E F G H I J K L M N O P Q
LEGEND
BEFORE 1st FILLING
AFTER 1st FILLING
AFTER 2nd FILLING
CLOSE
OPEN
SCALE
0 2 4 6 8 10 Cm
0 0,1 0,2 0,3 FT
VIEW SCALE
0 20 40 60 80 M
0 100 200 300 FT
NOTES :
– REFERENCE POINTS ARE ON BOTH SIDES OF THE HORIZONTAL JOINT AND NEAR IT.
– THIS IS A TRUE VIEW OF THE CONCRETE FACE.
• – MOVEMENT OF LESS THAN 3 mm (0,01 FT) OR NIL.
• – POINT DESTROYED.

b)–OPENING AND CLOSING OF VERTICAL JOINTS

FIG. 3.—MOVEMENT OF JOINTS

In the perimetral joints the most important movements, either for their numerical value or for their repercussions on the water-tightening system of the joints, are the relative movements of the slabs along the common joint. These are identified as sliding movements.

The movements in these joints were seen to be very slight before the first filling. After the first filling, with the exception of the movements of these joints between the vertical joints 12 to 20 whose values are of no significance in virtue of the raising of the joint P-Q, varying movements were noticed that were seen to be much more important than in the previous period.

The maximum values of the accumulated displacements from the beginning of the measurements were: on the left side—opening 3.0 cm (0.10 ft) and closing 1.8 cm (0.06 ft); on the right side—opening 4.5 cm (0.15 ft) and closing 3.2 cm (0.10 ft); sliding (between joints 22 and 23)—10 cm (0.33 ft) (Fig. 4(b)).

Before the first filling only very small movements of the connecting joints were noticed which do not deserve special mention. After the first filling, very small movements on the left side were seen and more considerable ones on the right side. The maximum accumulated displacements of opening and closing at this time were respectively 1.7 cm (0.06 ft) and 3.9 cm (0.13 ft) (Fig. 4(a)).

After the damages to the curtain were verified on the emptying of the reservoir in July, 1958, some repairs were carried out which included chiefly the following (Fig. 5):

1) Replacing the Guttaterna in some perimetral joints where the sticking was defective and was not able to bear the tension to which it was subject by the deformation of the joint, as well as in joint sections where the copper sheet and the Guttaterna yielded due to local deficiencies in the pouring.

2) The grouting without pressure, of cement and sand mortar in the space between the curtain and the placed rock beneath the uplifted perimetral slabs between joints 12 and 20. This was done through holes bored close together and at elevations successively higher, each hole being used only after having allowed the grouted mortar in the previous hole to set. These precautions were taken to prevent the increasing pressure from the mortar lifting the slabs even more and from being lost through the spaces in the placed rock. In this operation 495 cu m (647 cu yd) of sand were used and about 250 tons of cement.

3) The re-opening of some vertical joints to prevent the slabs from touching or becoming strained (Fig. 5). This was done by removing the concrete from one side of the joint making necessary, in consequence, the substitution of part of the copper sheet (the fold and one of the embedded parts) and the relaying of concrete at the edge of the slab leaving a further opening in the joint. In this repair 40 cu m (52 cu yd) of concrete were used.

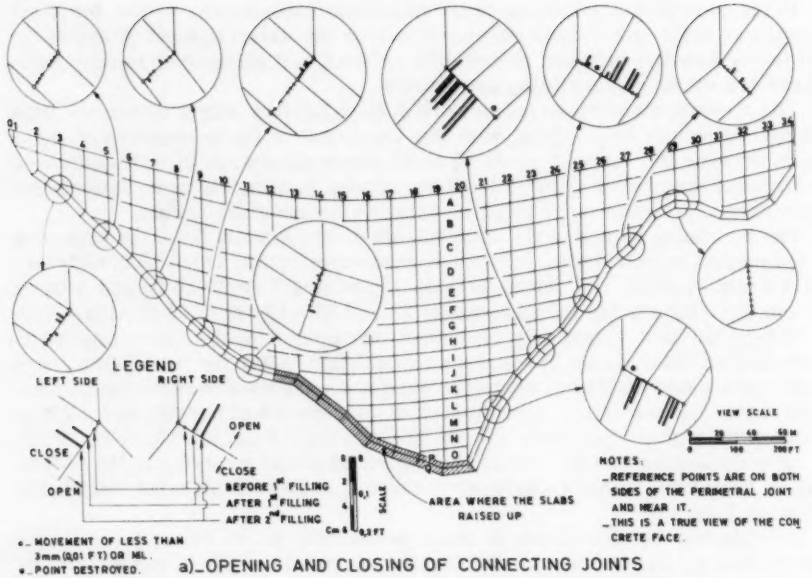
4) Complete replacing of the fold in the copper sheet of joint 26 that had opened a great deal and was found to be wrinkled by the relative movement of the slabs in the direction of the joint.

5) Repairs to a few local defects in pouring, especially near the copper sheets, and the replacing of some steps in the horizontal joints broken by stones that rolled down during the work on the rockfill of the upper part of the dam.

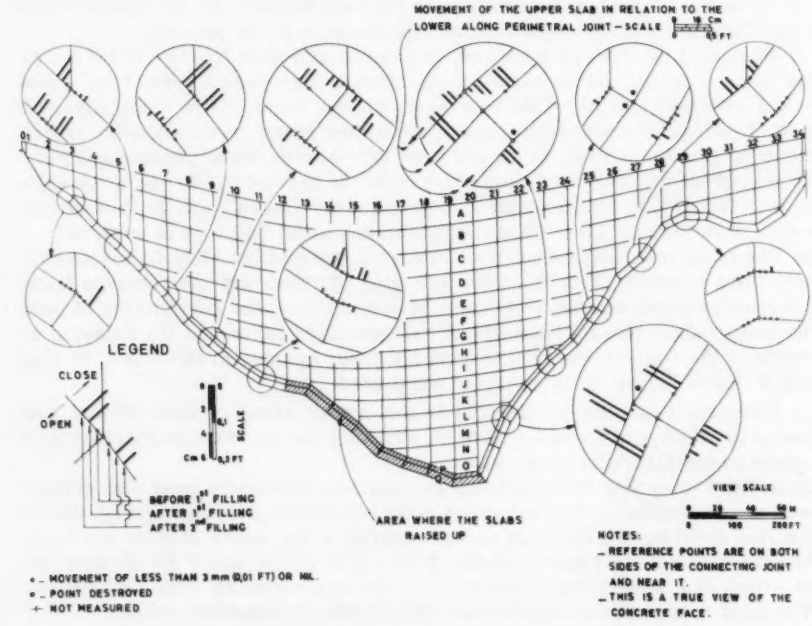
6) Filling cracks in slabs N 19-20, P 16-17, P 18-19, and P 19-20 with cement grout and superficial reclosing of the same cracks with Guttaterna.

The total cost of these repairs was 600,000$00 Escudos ($21,000).

Observations and Work Carried Out in 1959.—From the date when the elevation of approximately 720 m (2,362.3 ft) was reached, it was seen that the leakage began to be considerably greater than that registered in the previous



a)_OPENING AND CLOSING OF CONNECTING JOINTS


MOVEMENT OF THE UPPER SLAB IN RELATION TO THE LOWER ALONG PERIMETRAL JOINT – SCALE
LEGEND
CLOSE
OPEN
SCALE
BEFORE 1st FILLING
AFTER 1st FILLING
AFTER 2nd FILLING
AREA WHERE THE SLABS RAISED UP
VIEW SCALE
NOTES:
- REFERENCE POINTS ARE ON BOTH SIDES OF THE CONNECTING JOINT AND NEAR IT.
- THIS IS A TRUE VIEW OF THE CONCRETE FACE.
- MOVEMENT OF LESS THAN 3 mm (0,01 FT) OR NIL.
- POINT DESTROYED
← NOT MEASURED

b)_OPENING AND CLOSING OF PERIMETRAL JOINTS

FIG. 4.—MOVEMENT OF JOINTS

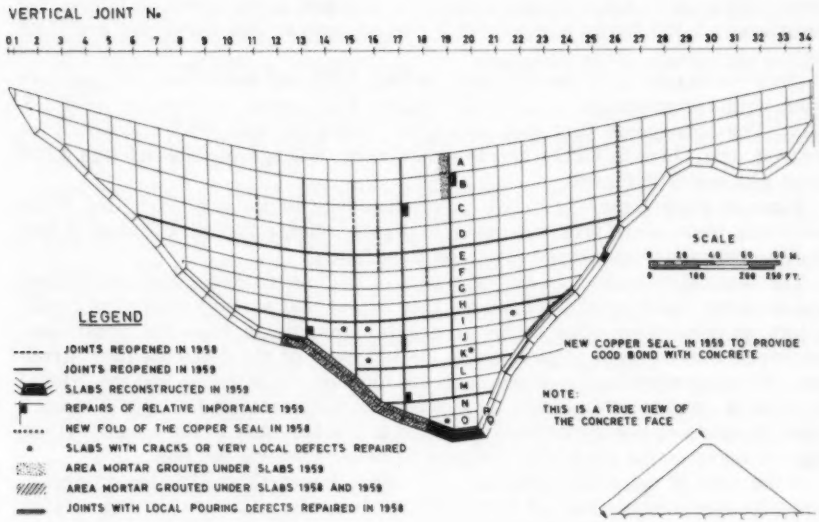

VERTICAL JOINT N°
01 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31 32 33 34
A
B
C
D
E
F
G
H
I
J
K*
L
M
N
O
SCALE
LEGEND
JOINTS REOPENED IN 1958
JOINTS REOPENED IN 1959
SLABS RECONSTRUCTED IN 1959
REPAIRS OF RELATIVE IMPORTANCE 1959
NEW FOLD OF THE COPPER SEAL IN 1958
SLABS WITH CRACKS OR VERY LOCAL DEFECTS REPAIRED
AREA MORTAR GROUTED UNDER SLABS 1959
AREA MORTAR GROUTED UNDER SLABS 1958 AND 1959
JOINTS WITH LOCAL POURING DEFECTS REPAIRED IN 1958
NEW COPPER SEAL IN 1959 TO PROVIDE GOOD BOND WITH CONCRETE
NOTE:
THIS IS A TRUE VIEW OF THE CONCRETE FACE

FIG. 5.—MAIN REPAIRS AND JOINTS REOPENED

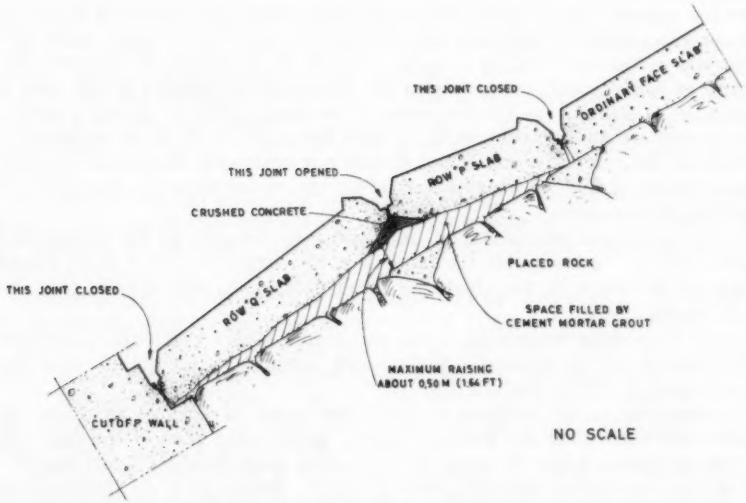


FIG. 6.—MOVEMENT OF PERIMETRAL SLABS

year for the same levels of water in the reservoir. For example, for the elevation 738.80 m (2,424.0 ft), the maximum reached in the previous filling, the leakage was 1,245 litres per sec (44.0 cfs) as against 382 litres per sec (13.5 cfs).

Until the beginning of the emptying in July, 1959, the water level varied very little, with the maximum of 740.80 m (2,430.6 ft) being reached on April 15, 1959. The leakage on this date was 2,740 litres per sec (96.7 cfs) which increased until June 3, 1959 when the maximum leakage registered was 3.020 litres per sec (106.6 cfs).

It can be seen from Fig. 1 that in consequence of the second filling of the reservoir, there were still considerable displacements in plan, although rather smaller than those measured after the first filling.

The general movement of the right part of the concrete face towards the left bank is particularly noticeable, so such an extent that at many points the right-to-left movement is seen to be of greater magnitude than the upstream-downstream movement. However, in the left part of the concrete face (from joint 15) the predominance of downstream movements is noted, and the lateral movements lead to settlements in a direction approaching the radial at each point. In addition, one's attention is called to the fact that in the second filling some of these points showed a tendency to settle towards the left.

In the area of the perimetral slabs, between joints 12 and 20, the upstream movement was emphasized, as referred to previously, as a result of the raising of the joint between slabs P and Q.

At the end of the second filling in 1959, the maximum absolute displacement downstream was of the order of 1.075 m (3.53 ft) and the maximum lateral displacements from the left to right was 0.169 m (0.55 ft) and from right to left 0.354 m (1.16 ft).

After the second filling, increases in the displacements in profile were observed, as was expected, but they were rather smaller than those registered up to the end of the first filling (Fig. 2). There is, however, an obvious predominance of the vertical component of the settlements, chiefly in the area to the right side of joint 15, in the increase of the displacements of many points.

In accordance with the movements in plan, the profiles show an increase in the raising of the joint of slabs P and Q, already noticed in that area in 1958, between profiles 12 and 20 which emphasizes the space between the concrete face and the placed rock.

The total maximum displacement, measured on the plan of the profile and in the direction of the normal to the face, is of the order of 2.04 m (6.69 ft) and the maximum deviation of the displaced point relative to that normal is about 0.39 m (1.28 ft).

The general closing movement of the horizontal joints was still registered after the second filling, although much more moderate and not altering the maximum shown in 1958 (Fig. 3(a)).

The movements of the vertical joints in the same directions as occurred previously continued after the second filling. In the wide area of the dam between vertical joints 4 and 22 there was a closing movement with varying increases whose maximum was of 4.9 cm (0.16 ft). Joints 23 to 29 continued to open in different proportions, with a maximum at joint 26 where there was an increase in opening of 3.8 cm (0.12 ft) thus raising the maximum opening of the joint to 13.2 cm (0.43 ft) (Fig. 3(b)).

After the second filling new movements were seen, in the perimetral joints making the maxima of the accumulated displacements to be: on the left side

an opening of 3.4 cm (0.11 ft) and closing 2.4 cm (0.08 ft); on the right side—opening 4.5 cm (0.15 ft) and a closing of 3.2 cm (0.10 ft); sliding (between joints 22 and 23)—12 cm (0.39 ft) (Fig. 4(b)).

After the second filling relatively small movements of the connecting joints were seen.

The maximum accumulated displacements were: on the left side an opening of 0.9 cm (0.03 ft); on the right side an opening of 2.3 cm (0.08 ft) and a closing of 4.5 cm (0.15 ft) (Fig. 4(a)).

In consequence of the relatively high leakages through the dam it was decided to empty the reservoir to inspect the concrete face, an operation which began on June 25, 1959 and ended on August 3, 1959.

Although sound detection equipment was used in the location of the leakages, when the concrete face was under pressure, the clear results that were hoped for were not obtained. It was the visual inspection, carried out by divers working about 10 m (33 ft) down, and completed by observation of the unwatered face that allowed the most important leakage points to be defined.

The most important damage was found to be in slab P 19-20 which showed a crack in the concrete along joint P-Q, that was about 3 m (10 ft) in length and openings on the surface varying between 5 cm and 8 cm (0.16 and 0.26 ft).

Besides this damage, to which the leakage shown can largely be attributed, more was found and can be described in brief as follows (Fig. 5):

1) A large number of the slabs showed almost capillary cracks, the majority of which were only superficial and did not penetrate the full thickness of the slabs.

2) On 4 slabs, B 19-20, C 17-18, I 13-14, and M 17-18, spalling of the layer of concrete above the plane of the reinforcement was observed, localized near the edges of the slabs and only affecting small areas. These damages were caused by the compressions that caused the closing of the joints.

3) On the slab Q 25-26 a crack about 2 cm (0.07 ft) wide on the surface completely cut off one of the corners of the slab.

4) In various stretches of the perimetral joints the Guttaterna was found to be detached from the concrete. This is explained by the opening movements of the joints and by the relative movements of the slabs along the perimetral joints.

5) The perimetral slabs between joints 18 and 20 were very fractured with indications of crushing caused by the compression of the upper slabs against the cut-off wall. It was in slab P 19-20 that the greatest leakage was discovered, as referred to previously.

6) The perimetral slabs between joints 12 and 18 showed an increase in the uplift already noticed the year before (1958) along joint P-Q, as they had become detached from the cement grouted in that year. This uplift was caused by the compression exerted by the slabs adjoining the perimetral slabs (Fig. 6).

7) The perimetral joint common to slabs P and Q 21-22 was misshapen in certain stretches as a result of the yielding of the copper seal due to local deficiencies in pouring around this seal.

8) The perimetral slab P 12-13, the appearance of which was normal except for the raising referred to previously, was broken when the reservoir was emptied about four days after it became unwatered. This occurrence is explained by the compressions already mentioned that increased with the expansion of the slabs under the influence of solar rays.

Most of the damages found were the result of compressions developed in the plane of the slabs because of the general deformation of the rockfill and the

closing of the joints, except for some damages that can be attributed to very localized construction deficiencies.

After analyzing the general deformations and the movement of the joints on the concrete-face, and the damages discovered, it was decided to allow new clearance in the joints that were found to be closed or almost closed, with a view to removing or relieving the compression between the slabs and to give greater possibility of settlement without breaking the slabs.

Because of the short time available between the completion of the emptying of the reservoir and the start of refilling, it was impossible to give more clearance to all the joints that had no clearance whatever, or very little, and a careful choice was made of those joints where widening would be more convenient. Thus, the vertical joints 13, 15, 17, and 19 were widened, except for some small stretches that maintained considerable clearance or had been widened the previous year.

The concrete on one side of the joint to be widened was destroyed and a new copper sheet was welded to the part embedded in the remaining side of the joint. The reinforcing that was exposed on the destroyed side was rebuilt, and repouring was accomplished. The total length of vertical joints that were widened was 674 m (2,211 ft). The experience gained in a similar operation the previous year (1958) showed that this repair is entirely efficient.

With regard to the horizontal joints, which from joint D-E downwards were almost all under pressure, it was decided to widen the joints D-E, H-I, K-L, and M-N, which gave a total length of 418 m (1,371 ft) of widened joints. A clearance of 5 cm (0.16 ft) was left in the joints. To fill in the joint, steam-packed cork board was used instead of wood.

Due to the compression between the slabs, special care was taken in beginning the widening process of the horizontal joints, because of the danger of important downward movements in the slabs immediately above the widened joint. This, in fact, did not happen because the movements as seen were normal. In any case, and for the sake of caution, the joints were successively widened from the top downwards.

The operation of widening the horizontal joints was carried out in a manner similar to that in the vertical joints. All the copper seals crossing-pieces were retained, because they were all found to be in good condition.

In filling the cuts that were made in the process of widening the vertical and horizontal joints, 312 cu m (408 cu yd) of concrete were used. At the same time as the work of widening the joints was in progress, repairs to the damage found in the concrete face were carried out. These were as follows (Fig. 5).

1) Narrow cracks in some slabs were reclosed where there were signs of leakage.

2) In the areas of slabs B 19-20, C 17-18, I 13-14, and M 17-18, that showed spalling of the concrete, the concrete was taken off locally until the upper mesh of the reinforcing was exposed and new concrete was poured.

3) In the corner of slab Q 25-26, detached by a crack, the concrete was removed and new concrete was poured.

4) The Guttaterna that had become detached in the perimetral joints was taken out and replaced in position.

5) The perimetral slabs between joints 18 and 20 and the perimetral slab P 12-13 were completely destroyed and new ones were made. In the pouring of these slabs 340 cu m (445 cu yd) of concrete was used.

6) In the perimetral slabs between joints 12 and 18, the underlying spaces were filled, grouting with cement and sand mortar without using pressure.

7) In the perimetral joint common to the slabs P and Q 21-22, the concrete at the edges of the joints was removed and a new copper sheet was placed in position and new concrete poured.

The removal of all of the concrete for the widening of the joints, or for the repair of damages, was carried out without the use of explosives.

Conclusions.—The observation of the upstream face of the Paradela dam, after the second filling and the complete emptying of the reservoir, shows that the concrete face of the dam, accompanying the movements of the rockfill, settled with movements that can be classified as: a) movements normal to the face of the dam; and b) movements tangential to the face of the dam.

Judging from present observations, the concrete-face showed completely satisfactory behavior relative to the movements normal to the face. With regard to the tangential movements, the behavior of the concrete-face was not entirely satisfactory however, as can be seen from the damages occurred in spite of the steps taken in the design with the intention of giving the concrete-face greater possibility to withstand these movements. These steps are seen to be completely justified and without them important damages would have occurred.

The tangential settlements of the concrete face at each point observed can be resolved into two components: one component occurs in the direction of the vertical joints and always downwards. The other component is in the direction normal to the other and directed from the right side towards the center, approximately in the two thirds—central and right—of the upstream face of the dam, and from the left side towards the center approximately in the left third.

The component of the settlement along the vertical joints tends to close the horizontal joints. This settlement is a consequence of the settlement of the rockfill in its direction because of the crushing of the contacts of the rocks, the breaking of the rocks, and the relative slidings of the rocks leading to a more stable arrangement. In dams constructed on a very deformable foundation, the difference of the settlements between the ground and the cut-off wall also helps to increase the component in question. Consequently, excepting the irregularities, the closing of a horizontal joint is greater when the height of the dam above the joint in consideration is greater, the rockfill material is less resistant, the construction of the rockfill in general and especially the placed rock is less efficient, and the foundation of the dam is more deformable.

At Paradela, where the horizontal joints were constructed with an opening of 3 cm (0.1 ft), filled with wood, to a depth of about 50 m (164.05 ft) no horizontal joint closed completely. Beyond this depth the general tendency was for the horizontal joints to close completely in the proportion allowed by the crushing of the wood between them. Once the joints were closed there was an accumulation of the forces on the lower slabs that caused the raising of the perimetral slabs between joints 12 and 20, as has been described.

As an important factor for future designs, it is pointed out that the total settlement of slab O between joints 19 and 20, and in the direction of these, reached 0.26 m (0.85 ft).

With regard to the component of the tangential settlements normal to the direction of the vertical joints it is seen that its direction is predominantly from right to left. This can be explained by the greater slope of the right side in relation to the left side, by the convex shape of the surface on which the rockfill lays on the right abutment, and by the greater deformability of the foundation of the left abutment, formed of heavily weathered granite.

Besides these causes, there may exist another much more important cause, namely the anisotropy of the rockfill resulting from the manner in which it was being dumped. But the writers haven't yet any firm ideas on this matter.

The opening and closing movements of the vertical joints, some even closing completely in certain stretches, are due to the component of the tangential settlements in question and to the variation of the horizontal length of the concrete-face because of the variation in the curvature.

Because the total of the tangential settlements has a direction more or less parallel to the perimetral joints on the right side and approximately normal to the perimetral joints on the left side, small opening and closing movements in those joints and important sliding movements in its direction were observed, while in the perimetral joints of the left side closing movements predominated.

Special attention is called to the important sliding movements seen along the perimetral joints of the right side. For example, in the perimetral joints situated between vertical joints 22 and 23 the total sliding movement in relation to the cut-off wall was 0.29 m (0.95 ft), distributed as follows: 0.10 m (0.33 ft) in the first perimetral joint; 0.12 m (0.39 ft) in the second; and 0.07 m (0.23 ft) in the joint with the cut-off wall. If it had not been for the existence of three perimetral joints and the precautions taken to guarantee the mobility and the water-tightness of the joints, serious damages could have taken place.

The experience gained from the performance of Paradela Dam until now can be summarized as follows:

1) The general distribution of the joints proved to be perfectly efficient.

2) The water-tightening joint arrangements functioned successfully. In spite of this, an attempt should be made to improve, even further, the arrangements of the perimetral joints in order to give better adaptability to the sliding movements along the joints.

3) The vertical joints in the central area of the dam should have greater initial opening to allow a greater closing movement. The vertical joints in the upper part of the right side should have been designed essentially for opening movements.

4) The horizontal joints should have been designed with increasing openings from joint F-G to a maximum of 0.07 or 0.08 m (0.23 or 0.26 ft) in the lower part of the dam and, in place of wood, a more compressible material, steam-packed cork board for example, should have been used. The same applies to the corresponding perimetral joints of the left side.

5) It is very important for the design of the joints to foresee the tangential settlements of the concrete-face, taking into account the slope, the configuration and deformability of the banks, the rockfill material characteristics and the anisotropy of the loose rockfill according to the carrying out of the dumping.

The study of such anisotropy and the making of tests on the rock utilized in large dams constructed by similar processes is of great interest in order to get information on the influence of the rock characteristics on the yielding of rockfills, and, thus, to be able to foresee more certainly the settlements of the concrete face.

6) The slope 1.3-on-1 of the upstream face, the relatively large thickness of the placed rock wall, and the very careful laying of this wall were factors that contributed greatly to the successful behavior of Paradela Dam. In fact, if the upstream face had been given greater slope and a thinner placed rock wall, or one less carefully layed, the tangential settlements of the concrete face would have been greater, especially in the direction of the vertical joints, either because the stresses in the wall would be greater or because its deformability would have increased. Under these conditions the movements of

the joints would have been greater and the resulting damages would have been much more important.

7) The adoption of a relatively important curvature at Paradela has advantages and disadvantages. It is possible to eliminate the disadvantages.

In effect, the existence of the curvature causes the formation of transversal arches inside the rockfill whose deformation tends to intensify the lateral contacts between rocks along the development line of the arches. The intensification of these contacts, thus increasing the lateral compression, makes the settlements along the line of the vertical joints, as well as the slidings along the perimetral joints, a little more difficult. These are the desired advantages and it is believed that they have been obtained although they cannot be appreciated quantitatively.

On the other hand the increase of lateral compression resulting from the shortening of the arches facilitates the closing of the vertical and perimetral joints when the latter tend to close. This occurs with the perimetral joints in the left side of Paradela dam. These are the disadvantages of the curvature, disadvantages that can be controlled by giving those joints greater possibilities of closing. The expression "arch action" is purely comparative but it is used to give an approximate idea of what really takes place.

UNDERGROUND POWER STATIONS IN ITALY[a]

Discussion by F. L. Lawton

F. L. LAWTON,[1] M. ASCE.—The authors have provided a most comprehensive and valuable presentation on the outstanding part played by underground power stations in Italian hydroelectric development. The importance of this is emphasized by the fact that 33% of the installed hydroelectric capacity and 26% of total generating capacity is contained in underground stations. The significance of this is further stressed by the fact that 30% of Italian hydroelectric capacity is contained in the 57 stations treated in the paper, these stations numbering only 2% of the total of 2,750 hydro plants.

The technical and economic justification for the adoption of underground stations in Italy is clearly stated by the authors. It is interesting to note the emphasis placed on the point now generally regarded as the basis for selecting or discarding the underground solution: that is, "- - general layout of the scheme." There is no question as to the soundness of this concept. A decision cannot be reached on the economic effect of a shorter penstock alone, for instance, because such a layout is generally accompanied by longer tailrace tunnels, access tunnels, etc.

The authors' statement to the effect that "it should be underlined that the tendencies to leave rock blocks within the masonry for economy's sake has to be regarded with caution. Even if the rock is solid, the discontinuity in the planes of foundation may cause small settlements and fissures clearly unfavorable to the foundations of the turbine-generator sets" is important. It is sometimes overlooked.

The observations on excavation methods quite correctly noted the significance of the orientation of the machine chamber with respect to the strike and dip of rock strata. The longitudinal axis should be so orientated, if at all possible, as to cross the strata at right angles.

Adequate preliminary subsurface investigations are imperative. These may consist of test tunnels so located as to be usable in the permanent construction, or simply diamond drill holes. However, the data should be presented in such a form as to clearly show the geological structure in block form, because only in this manner can it be most effectively interpreted by the design and construction engineers.

There appears to be no doubt geology plays an unusually important part in Italian underground power plants and is fully taken into account in design and construction. Does seismic activity influence the design?

Fig. 6 of the paper is an excellent presentation of the cross-sectional shapes and lining features of the principal underground power stations in Italy. It is

[a] December, 1959, by Dante Finzi, Mario Mainardis, and Carlo Semenza.

[1] Vice-Pres. and Chf. Engr., Power Div., Aluminium Labs., Ltd., Montreal, Canada.

unfortunate that the sections are difficult to interpret due to the small scale. This is especially true in the case of the illustrations for Monte Argento on the one hand and Quart, Signayes, and Coghinas on the other.

It would have added to the value of the paper if the authors had said something about the long-term adequacy of the Bressanone 6 m diameter concrete penstock for a maximum head of 164 m and similar Italian construction.

One of the interesting aspects of the many stations dealt with in the paper, probably due to topographic and geological conditions, involves the unusually complex access supply and discharge tunnels associated with the station.

In view of the great influence of the geological conditions on many of the Italian underground stations and the ingenious designs resulting therefrom, it is to be regretted that the authors did not include some relative cost data.

DETERMINATION OF STRESSES ON ANCHOR BLOCKS[a]

Discussion by Simon Peters

SIMON PETERS,[1] F. ASCE.—A very comprehensive paper of particular interest to civil engineers engaged in the design of hydroelectric developments has been presented. The fact that Mr. Bouchayer's method is stated to allow savings up to 75% of the cost of penstock anchors, compared with the usual anchor design, makes this paper of special significance. The paper and this discussion are confined to rigid field-jointed pipe, and do not consider the increasingly used flexible coupled lines.

Methods of Design.—There are two methods of anchor design: one, called the "Swiss method," is to embed each bend in the pipeline in a concrete anchor and to place an expansion joint between anchors; the other, called the "French method" considers part of the pipeline, including one or more bends, as an arch and anchors only the abutments of the arch. The penstock in this latter case is a continuously welded pipe without expansion joints. The usual practice in the United States is to follow the Swiss method.

The basic difference between the two design methods is the concept of the function of the pipe. While in the Swiss (and American) method, the pipe is considered a container for the purpose of withstanding the pressure of the water with as little structural duty as possible (beam action and compression strut), in the French method the functions of container and structural member are combined. The pipe thus becomes a highly stressed structural member and is designed to resist much greater bending moment and compressive stresses, mostly as an arch with fixed ends.

Expansion Joints.—The Swiss method has the advantage of simplicity of design and results in lighter-weight pipe. The distinctive feature of the Swiss method is the presence of an expansion joint between two anchors. The expansion joint plays a very important role and fulfills the following three functions:

1. It reduces the temperature stresses in the pipe to zero, except for the small forces due to the friction at the gland and water pressure on the end of the pipe shell.
2. The expansion joint has the function of "make-up joint," because it is easily collapsable and is the last section to be placed between anchors.
3. It permits construction from number of locations at the same time, rather than from only one location.

Limitations of French Method.—The French method can be applied only to a continuously welded pipe located on sufficiently undulating terrain to permit arch action. It cannot be used with a banded penstock because the pipe shell

[a] December, 1959, by M. R. Bouchayer.

[1] Civ. Engr., Pacific Gas and Elec. Co., San Francisco, Calif.

is usually too thin to support arch action. It would be difficult to use this method in a penstock having horizontal bends since it would involve complicated intermediate supports to limit lateral deflection of the pipe. In all such cases at least part of the pipe must be anchored in accordance with the Swiss practice.

Another complication inherent in the French method is the difficulty of assembly. The last section of the pipe must be cut to the exact length of the gap, measured at a certain temperature, and the actual tailoring of the section done in the field. Then a colder temperature is awaited in order to roll the section into the gap. The joints will close as soon as the temperature rises, after which they should be quickly tack-welded in order to "freeze" the section in place; regular welding follows. If it is remembered that penstocks are often located in rugged, steep terrain and assembled at extreme temperatures with one side of the penstock often much colder than another and with difficult conditions for movement of workers on and off the job, one can see the complications of this method in comparison to Swiss penstock practice. If, in addition to the difficulties outlined, one can visualize that the arched pipe would have the tendency to leave the intermediate supports and lift itself up during assembly, it will be realized that the installation of such a penstock could be entrusted only to highly technical personnel.

It is interesting to note that while some French engineers favor a rigid, fixed-ended arch design for their penstocks, the practice in the United States has a tendency to develop in the opposite direction toward relaxed, hinged pipe assembly by the introduction of the Dresser Couplings or similar flexible joints. This gives the penstock flexibility, reduces the number of anchor blocks by taking small deflections in the joints, and has the advantage of speed and simplicity of field assembly. There is no doubt that higher labor costs in the United States give impetus to flexible joints preference.

Possible Concrete Saving in Swiss Method.—The author cites an example of a penstock designed by Swiss and French methods. The volume of concrete in the anchors designed by the Swiss method is four times that of French design with computations based on 2 tons per sq ft of allowable bearing pressure.

This is a very substantial savings of concrete. However, in reviewing the example, one will notice that a pressure of 2 tons per sq ft is a very low figure, and since penstocks are often located on rock ridges, a much higher value would be used in these cases, with the result that the size and the volume of the anchor would be then governed by considerations other than bearing pressure. Thus, the volume difference in the design of the two methods would be narrowed down.

Another interesting consideration that reduces the volume of concrete in anchors designed in the United States is the fact that penstocks are often embedded up to horizontal diameter only. As Fig. 1 shows, the volume of concrete from the "cap" of French design, could be very advantageously used for widening or otherwise enlarging the base and thus reducing the pressure materially. This method has a further advantage of lowering the center of gravity of the anchor, thus improving the stability of rotation and moving the center of gravity upstream which improves distribution of the pressure.

A Design Practice.—The design considerations that govern the size and volume of penstock anchors, as adopted and used by Pacific Gas and Electric Company, are as follows:

After the resultant of all the forces (except dead weight of pipe plus weight of water and dead weight of anchor) has been determined, it is resolved into

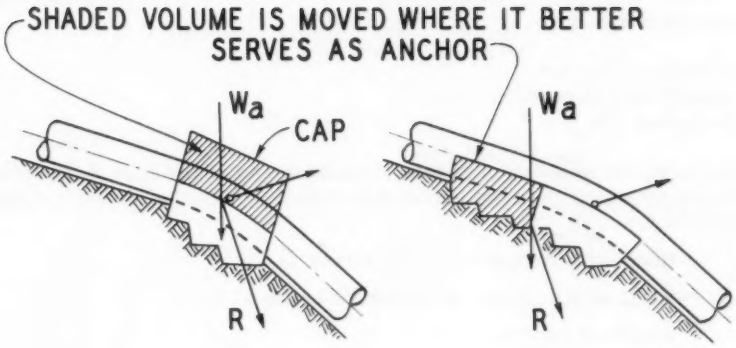


FIG. 1

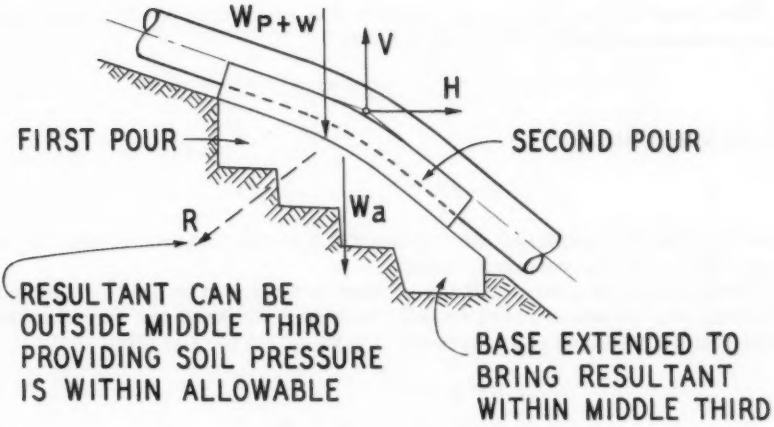


FIG. 2

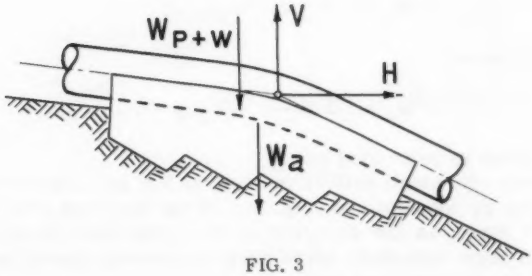


FIG. 3

horizontal and vertical components applied at the point of intersection of tangents.

The anchor must satisfy the following three conditions of stability with adequate factors of safety:

1. Sliding equilibrium;
2. Equilibrium of rotation; and
3. Soil pressure.

Sliding equilibrium determines the minimum weight of the anchor. By using the following notations, the equation for minimum weight of the anchor is easily deducted (Fig. 2).

H = Horizontal component of the forces acting at P.I.

V = Vertical component of the forces acting at P.I.

W_a = Weight of anchor.

W_{p+w} = Weight of pipe and water, supported by anchor.

F = Sum of all vertical forces acting on anchor.

The term V can be positive or negative. It is considered positive when acting downward and negative when causing uplift:

$$F = \pm V + W_a + W_{p+w} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

For sliding stability,

$$H = fF \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2)$$

in which f is the coefficient of friction (f = 0.35 for concrete on wet clay and sand and f = 0.65 for concrete on rock).

Design drawings always call for anchors to be placed on solid, undisturbed material. For locations where no solid rock is in evidence and sampling indicates that clay or sand is not present, a value of 0.5 for f is often used:

$$H = 0.5\,F \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3)$$

and

$$H = 0.5\left(\pm V + W_a + W_{p+w}\right) \quad \ldots\ldots\ldots\ldots \quad (4)$$

Therefore, the weight of the anchor is

$$W_a = 2\,H \pm V - W_{p+w} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (5)$$

which will be changed to

$$W_a = 1.5\,H \pm V - W_{p+w} \quad \ldots\ldots\ldots\ldots\ldots \quad (6)$$

if foundation is known to be in good rock.

Anchors on rock with small deflection angle at P.I. are often designed with base roughened up by indentations (Fig. 3). These keyways give extra horizontal resistance force, as the concrete or the rock must shear before the sliding factor will come into play. Since shear resistance cannot be evaluated with certainty, it is not taken into account.

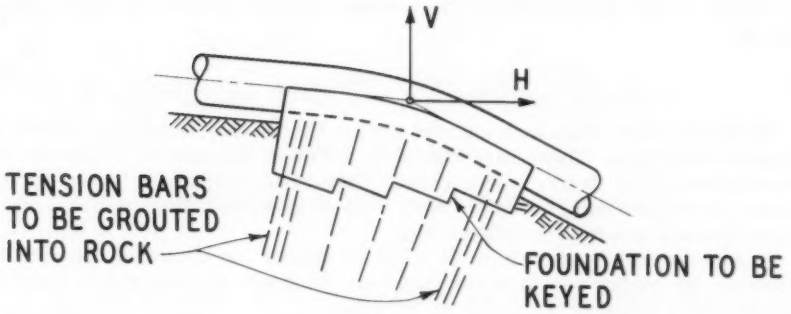


FIG. 4

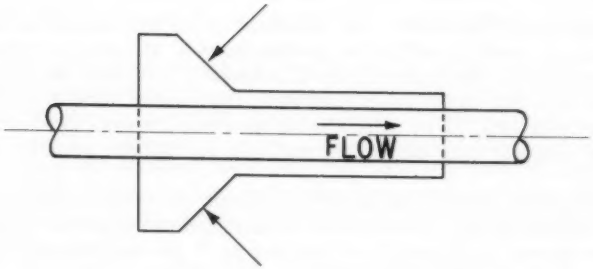


FIG. 5

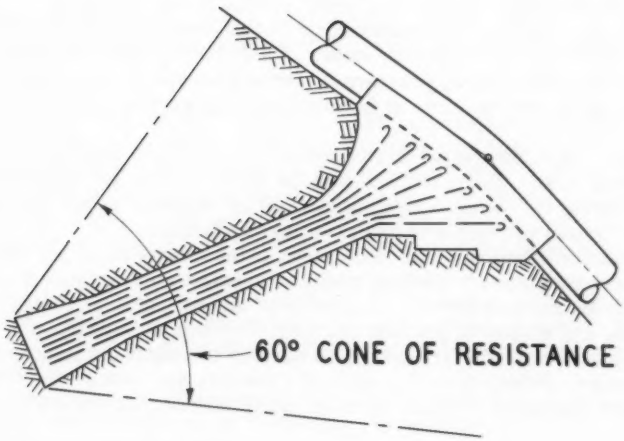


FIG. 6

Weight of the anchor should be checked further against excessive uplift forces.

$$W_a = 1.5 \left(V - W_{p+w}\right) \ldots\ldots\ldots\ldots\ldots\ldots \quad (7)$$

Whenever solid rock is found, the anchor can be tied to rock by means of grouted steel bars. These bars should be computed and spaced to provide for tension forces for both expanding and contracting conditions. Since forces tending to cause sliding cannot be taken care of by shear in the steel bars, adequate keyways are to be furnished (Fig. 4).

In cases where V is positive, it helps to reduce the weight of the anchor. However, the anchor should be checked for sliding with the pipe empty and with forces due to temperature acting on the pipe.

In a recent installation, an anchor was poured in the form of a letter T creating two wing walls. This resulted in considerable saving of the concrete, by calling on the passive resistance of the earth to help in sliding equilibrium (Fig. 5).

Equilibrium of Rotation.—The resultant of all the forces (including weight of the anchor) must lie within the middle third of the anchor base. To satisfy this condition, the first pour (Fig. 2) is usually extended downstream. When the resultant tends to intersect outside of the middle third in an upstream and uphill direction (Fig. 2), no provision to increase the base is provided for since it is very unlikely that the anchor will roll up against passive resistance of the bank.

Soil-pressure limitations serve to control the overturning tendency, since overturning cannot take place before crushing of the material takes place. Soil pressure should be limited to 6 kips per sq ft for undisturbed firm material and 30 kips per sq ft for solid rock.

General.— Preferably the pipe should be designed so that no field joint will be encased in the concrete anchor. It is desirable to have the upstream and downstream sides of the anchor (top pour) normal to the pipe axis.

In two recent penstock installations, there has been introduced a novel and economical anchoring of the pipe by means of a drift excavation filled with concrete, the holding force being provided by large diameter tension bars, transmitting the tension into the cone of firm decomposed granite or weathered rock (Fig. 6).

Conclusion.—Mr. Bouchayer has introduced a novel and ingenius method of anchoring penstocks. Full credit should be given to him for demonstrating that forces acting on the arch system are not as large as might be expected and that the system is readily within practical application.

This discussion points out that the savings in the volume of anchor concrete, claimed in Mr. Bouchayer's method, would be considerably reduced if conventionally designed anchors (Swiss and American methods of design) were carefully analyzed and shaped to provide the most economical solution.

The example cited in the article shows savings in concrete volume when the governing design criteria is a low value of allowable soil pressure. This discussion shows that anchor weight is principally determined by sliding equilibrium.

It has been further pointed out that Mr. Bouchayer's method has its limitations, namely, it cannot be used on (1) straight pipe, (2) banded penstocks, (3) penstocks with horizontal angles, and (4) flexible coupled penstocks.

AN UNUSUAL DEPOSIT IN A HYDRAULIC TUNNEL[a]

Discussion by F. L. Lawton and William T. Walker

F. L. LAWTON,[1] M. ASCE.—The authors are to be congratulated on a fine contribution to the sparse literature on the subject of deposits in power tunnels and their significant effect in respect of hydraulic head losses and accompanying economic losses.

Their findings show that a relatively thin deposit on tunnel walls can occasion substantial energy losses in a hydroelectric plant supplied through a long tunnel.

Reference has been made to the sparse literature on the subject. There is no doubt many power developments involving both lined and unlined supply tunnels incur friction losses in excess of design value. These excess losses arise from many causes, among which may be noted deposits of the type encountered at Santeetlah, insect infestations, algal growths, and mud deposits. Sometimes, in the case of unlined tunnels which have been in service for many years, rock fragments accumulate on the invert as a result of rock falls from the arch and walls.

Operating organizations generally carry on observations which show the increased head losses and, occasionally, make inspections of tunnel conditions. However, all too frequently the findings are filed and never made available to the engineering profession, thus limiting the development of the knowledge necessary to advance the actual performance of new tunnels. A plea is made for the presentation to the engineering profession of papers dealing with this subject.

WILLIAM T. WALKER,[2] M. ASCE.—The profession is indedted to Messrs. Pollard and House for the contribution they have made toward a more complete knowledge of fouling deposits in power conduits. Their paper is of particular interest to the writer because, as an associate of Mr. House, he had the opportunity to observe the solution of the Santeetlah head-loss problem from initial discovery of the power loss involved to removal of the deposit with hydraulic jets.

The writer can add nothing important to the paper concerning the work at Santeetlah. However, the following brief description of experience with this type of tunnel deposit at Nantahala Thorpe hydroelectric developments may be of interest.

A large percentage of the pressure conduit at Nantahala and Thorpe consists of unlined tunnel whereas at Santeetlah there is no unlined tunnel. There is some indication that the hydraulic characteristics of these unlined tunnels have

[a] December, 1959, by Arthur Lewis Pollard and Hazen Etnier House.
[1] Vice-Pres. and Chf. Engr., Power Div., Aluminium Labs., Ltd., Montreal, Canada.
[2] Chf. Engr., Natahala Power and Light Co., N. C.

been improved by this deposit whereas at Santeetlah, with no unlined tunnel, the head loss continues to increase with thicker deposits.

Nantahala Dam is located in Macon County, N. C. at mile 22.8 on the Nantahala River, a tributary of the Little Tennessee River. Water is supplied to the power house which is located at mile 13.6 through a pressure conduit 29,654 ft long. Over 84% is unlined tunnel 13 ft by 14 ft, 14 ft by 13 ft, 12 ft by 12 ft. Except for a negligible amount of concrete-lined tunnel, the remainder consists of steel-lined tunnels and exposed steel pipe with welded joints varying in diameter from 10 ft to 8 ft. All steel is coated with bitumastic enamel. The maximum gross head with the lake full is 1,008.3 ft. Minimum gross head is 877 ft.

During a test run in 1942, shortly after the plant was first placed in operation, the head loss from the reservoir to the turbine when the unit was discharging 600 cfs was found to be 37.0 ft. The generator output at a gross head of 1,000 ft when discharging 600 cfs was 44,700 kw.

Thorpe Dam is also on a Little Tennessee tributary in Jackson County, N. C. at mile 9.7 on the West Fork of the Tuckasegee River. The power conduit which supplies the power house at mile 3.4 of the same river is 16,287 ft long. More than one third of its length is 12 ft by 12 ft unlined tunnel; less than 200 ft is 10 ft diameter concrete lined and the remainder consists of welded-steel tunnel lining and exposed pipe varying in diameter from 10 ft to 6 ft. All steel is coated with bitumastic enamel. The maximum gross head at full lake is 1,207.3 ft. Minimum gross head is 1,130.5 ft.

TABLE 1

Date	Tunnel No. 1 (Unlined)	Tunnel No. 2 (Unlined)	Tunnel No. 3 (Steel Pipe)
1950	3/8"	3/8" - 1/4"	3/16" - 1/16"
1954	-	-	1/4"
1959	3/4"	1/2"	3/8"

The plant was placed in service in 1941 and tests similar to those at Nantahala were made in 1942. The generator output at a gross head of 1,190 ft when discharging 260 cfs was 22,000 kw. The head loss from the reservoir to the turbine was 16.0 ft.

It is not known exactly when the black tunnel deposit, which now coats the interior of Nantahala and Thorpe conduits, first appeared. However, it was observed and the thickness measured during inspections of 1947 and 1948.

At Nantahala the thickness in 1948 was 1/6 in. in exposed pipe sections, 1/8 in. in underground pipe and 1/4 in. to 3/8 in. in unlined tunnel sections. Now (1960) the deposit averages approximately 1/2 in. thick in unlined tunnels and 3/8 in. in pipe sections.

At Thorpe the thickness was estimated to be 1/4 in. in 1947. Measurements made later are shown in Table 1.

The Nantahala head loss from reservoir to turbine at a discharge of 600 cfs increased approximately 2.5 ft from 1942 to 1949 and has not changed appreciably since. In fact recent tests indicates about the same loss as the 1942 test.

The loss at Thorpe increased approximately 7 ft to 9 ft from 1942 to 1949 and recent tests indicate no appreciable change since 1949.

It is assumed that the negligible increase in conduit losses at Nantahala and the small increase at Thorpe as compared to Santeetlah may be attributed to the lack of an appreciable quantity of concrete tunnel lining and the large percentage of unlined tunnel at each project (84% at Nantahala and 36% at Thorpe). This assumption is supported by the test results at Appalachia tunnel where a similar manganese coating resulted in a decrease in friction coefficient for all surfaces except concrete lining and exposed steel pipe where the manganese coating was roughened when flakes of the deposit fell off while the conduit was unwatered.[3]

[3] "Friction Measurements in the Appalachia Tunnel," by Rex A. Elder, Transactions, ASCE, Vol. 123, 1958, p. 1249.

SIMPLIFIED PROCEDURE FOR STRESS ANALYSIS OF GRAVITY DAMS[a]

Discussion by G. S. Sarkaria

G. S. SARKARIA,[1] M. ASCE.—The use of coefficients (which are independent of the geometry of the cross section of a gravity dam) for obtaining internal stresses is indeed a time-saving procedure. The limitations within which the method is correctly applicable have been mentioned by Mr. Sandhu in his introduction and again in the assumptions. It is interesting to study the effects of these limitations on the stresses in a gravity dam.

Mr. Sandhu has pointed out that in gravity dams which function as three-dimensional structures because the transverse contraction joints are either keyed and/or grouted, results of the gravity method of analysis do not truly reflect the stress conditions in the dam. However, once magnitude of the portion of the external load which is sustained by vertical elements of the dam is determined, as for example by the trial-load method of analysis, the principal stresses in a transverse vertical plane can be obtained by following the author's method. The vertical element's share of the load would then be substituted for the external pressures P and P'. The results would still be approximations, since it is assumed that σ_x and the tangential shear stresses are zero, which is not true in a three-dimensional monolithic structure. But these results would generally be acceptable for the design of reinforcement around openings in the dam.

Examination of designs of various gravity dams shows that the height limit for economical design of cantilever gravity dams is about 450 ft. The highest dams of this type are Detroit (450 ft) and Pine Flat (440 ft), both of which are in the United States. At comparatively narrow sites with steep abutments where substantial lateral transfer of load to the abutments can be realized, it is desirable to key and grout the transverse joints, because the economy in concrete thus achieved would offset the cost of forming keyways and grouting the joints. This would be more so for gravity dams higher than about 450 ft. Therefore, the gravity method of analysis including the author's simplifications, are generally applicable to gravity dams lower than 450 ft only.

The assumption regarding linear variation of vertical normal stresses is relatively true only for gravity dam elements which have no longitudinal contraction joints and the deformations can be assumed to comply with the premises of simple flexure. Gravity dams can be built to a height of approximately 450 ft without providing longitudinal contraction joints, and then also with rigid control of heat of hydration. For dams higher than 450 ft, the influence of the longitudinal joints and the schedule for grouting them should be taken into consideration. It has been shown[2] that vertical normal stresses (σ_z) so obtained

[a] December, 1959, by R. S. Sandhu.

[1] Supervisory Engr., Internatl. Engrg. Co., Inc., San Francisco, Calif.

[2] "Influence of Construction Program on Stresses in a Large Concrete Dam," by G. S. Sarkaria, Central Board of Irrigation Journal, India, May, 1950.

vary considerably from those obtained for the dam without longitudinal joints and assuming linear variation. In Fig. 1, results of stress analysis of a high gravity dam are shown. External loads consist of full reservoir pressure and effects of horizontal and vertical earthquake. A gallery, 100 ft wide and 150 ft high, is located in the body of the dam. Fig. 1(a) shows σ_z stresses at three elevations computed by the gravity method of analysis, and showing linear distribution as well as the effect of longitudinal joint grouting program. Fig. 1(b) shows the stress at the same locations, but determined by the trial-load method including the effect of lateral transfer of load to abutments. All these stresses

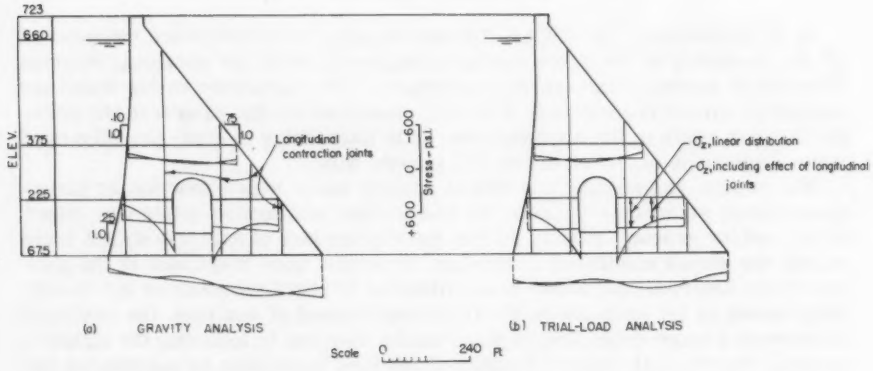


FIG. 1

were computed without including effect of hydraulic uplift in the dam, and are all compressive. From these results it is seen that:

(a) The longitudinal contraction joints and the program according to which monolithic action is established in the transverse direction, have considerable influence on the vertical normal stresses.

(b) Location of a large longitudinal gallery in the dam radically alters the stress distribution.

(c) Three-dimensional monolithic action in dam reduces the load on the vertical elements, thereby improving the compressive vertical normal stress at the upstream face.

The first two conclusions show that the assumption of linear distribution of σ_z—stresses would not be justified, and the principal stresses calculated by the author's method would be entirely different from those based on σ_z stress as calculated previously. In such cases it would be necessary to use the relationships that express the shear and principal stresses in terms of the vertical normal stress.

ERRATA

Journal of the Power Division

Proceedings of the American Society of Civil Engineers

December, 1959

p. 63. At the end of the line 1 of the SYNOPSIS add the words "and is still "

p. 63. In line 3 of the main body of the paper insert the date June, 1957 in parentheses after the word "present."

p. 64. At the end of the fifth full paragraph insert the following paragraph:

It should be noted here that in evaluating the importance of each plant in Italy, one should not consider its capacity only As is well known, in Europe, and particularly in Italy, to obtain capacities similar to those installed in many American plants, it is often necessary to sum up to 10 to 20 hydroelectric plants. In fact, in Italy plants from 50,000 to 100,000 kw are above the normal average. On the other hand, it is evident that the total work required for the planning and building of a plant is by no means proportional to the kilowatts installed; the hardest problems are often met when dealing with small plants. That means that to obtain a capacity similar to that of many of American plants, we must often undertake a much more extensive and engaging work; which, of course, also results in the gain of a comparatively wider experience.

p. 65. The title for the illustration should be Fig. 2.

p. 75. At the end of the second full paragraph insert the following paragraph:

From the above, there obviously results the convenience of considering the ensemble of the adjoining structures of a power station (from surge tanks to the outlet of the discharge canal included) in order to estimate which of the solutions—underground or aboveground—is more economical.

p. 76. In Fig. 5 interchange the subheadings c and d.

p. 87. In the second full paragraph, at the end of the first sentence, insert the following sentence:

Besides, geologic conditions are often not very favorable to the adoption of the scheme A, due to the rather poor qualities of the rocks and the water infiltration dangers; so that the scheme B is almost generally adopted.

p. 88. In item (3) change "Belvisio" to "Belviso."

p. 91. In the second full paragraph, in line 1, change "owing to the natural geological . ." to "owing to the prevalent natural geological . . ."

p. 91. In the third full paragraph, in the last line, change "thrusting or difficult rock" to "thrusting of other types of difficult rock."

p. 91. In the fourth full paragraph, in line 4, delete the word "rock" at the end.

p. 96. In the first full paragraph, in line 4, delete the first word "power" and substitute therefore the word "capacity."

p. 99. Add the following to the end of the ACKNOWLEDGMENTS: "and especially to Ennie Gallo who helped the authors in drawing up these notes."

p. 173. In line 3 of the third paragraph under the heading "Introduction" change the first word "of" to "or."

p. 174. Next to the heading "Notation" insert, in parentheses, "See Fig. 1."

p. 176. In line 1 of the third paragraph change "segment ABCC' A' " to "segment ABCC' B' A' ." In Eq. 1 change all K' s to k' s.

p. 176. In Fig. III change P_E to P_e' and change $Z\tau$ and reverse the arrows for σ_y and σ_z.

p. 177. In line 4 change "to a horizontal earthquake increasing the . ." to "to a horizontal earthquake giving relative downstream acceleration $\alpha_e g$ and a vertical earthquake increasing the . . ." In line 11 change the term $_{AA'}\int^{BB'}$ dz to $_{AA'}\int^{BB'}\tau dz/_{\circ}$ and in line 12 change ACC' $_{A'}$ to ACC' A' . In line 14 add the term τ before the equals sign. In line 16 change K to k. In line 23 change λ^+ to λ^*.

p. 178. In line 1 of Eq. 3 change λ^+ to λ and in line 2 of Eq. 3 change λw^+ to $\lambda^* w$. In line 7 change $\frac{y\,W}{2T^2}$ to y W/ T^2 and in line 8 insert brackets after the term 6 M/T^2 and before dT /T dz. In line 17 change M/T to W/T.

p. 179. In line 2 change 2k = 1 to 2k - 1 and in line 6 insert a parentheses before the term 1 - 3k. In line 9 change P' to P'_e and in line 10 change P to P_e. In line 17 change λw^+ to $\lambda^* w$.

p. 180. In line 5 change the term k^2 to k and in line 10 insert a bracket following the term tan ϕ_u. In line 11 change W_e to W_c and in line 14 change $\lambda wh'$ to / λwh. Also in line 14 change K^2 to k^2.

p. 181. In line 2 change K to k and in line 26 insert a plus sign prior to the term dP'_e/dz.

p. 182. In line 1 insert a parentheses after the term $1 - k^2$ and in line 17 change $C_5 V/T$ to C_6/T.

p. 183. In line 1 change K to k and in line 4 add an equals sign at the start of the line. In line 14 change k_2 (1 - k) to $k_{2(1 - k)}$.

p. 184. In line 3 the two letters C should be changed to C_8 and C_9 / and in line 10 the asterick that follows the bracket should be inserted prior to the bracket and immediately after the parentheses. In Col. 1 of Table 1 change K to k.

p. 185. In line 1 change $4k_2 \tan \phi$ to $4k_2 \tan \phi_u$.

p. 186. In Col. 1, Table II change K to k and in this same column insert > signs following the letters z.

p. 187. In Col. 1, Table III change K to k.

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p. 65. Through an oversight, the following illustrations were not included with John H. Ruskin's discussion:

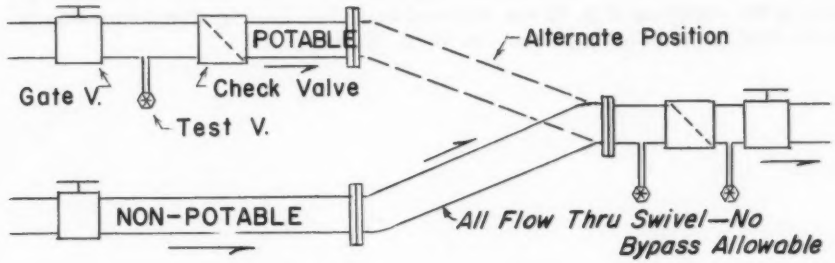

FIG. 1.—SIMPLE LAYOUT FOR SWIVEL CONNECTION

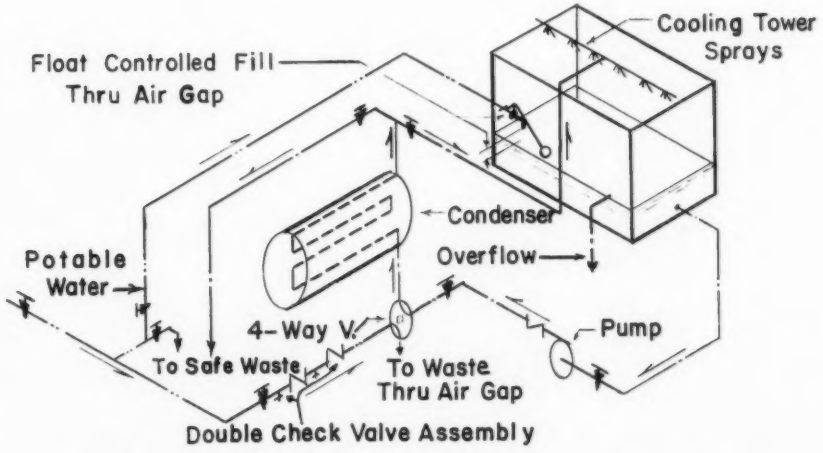

FIG. 2.—SCHEMATIC LAYOUT OF COOLING WATER SYSTEM INCORPORATING FOUR-WAY PLUG VALVE

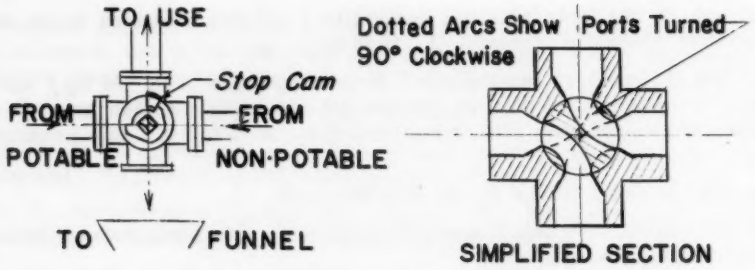

FIG. 3.—DETAILS OF FOUR-WAY PLUG VALVE

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p. 71. The material that is apparantly missing from between the end of p. 71 and the beginning of p. 72 can be found on p. 70. That is, this discussion is to be read in the following order: p. 71, p. 70, and p. 72.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 2270 is identified as 2270(ST9) which indicates that the paper is contained in the ninth issue of the Journal of the Structural Division during 1959.

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JUNE: 2048(CP1), 2049(CP1), 2050(CP1), 2051(CP1), 2052(CP1), 2053(CP1), 2054(CP1), 2055(CP1), 2056(HY6), 2057(HY6), 2058(HY6), 2059(IR2), 2060(IR2), 2061(PO3), 2062(SM3), 2063(SM3), 2064(SM3), 2065(ST6), 2066(WW2), 2067(WW2), 2068(WW2), 2069(WW2), 2070(WW2), 2071(WW2), 2072(CP1)[c], 2073(IR2)[c], 2074(PO3)[c], 2075(ST6)[c], 2076(HY6)[c], 2077(SM3)[c], 2078(WW2)[c].

JULY: 2079(HY7), 2080(HY7), 2081(HY7), 2082(HY7), 2083(HY7), 2084(HY7), 2085(HY7), 2086(SA4), 2087(SA4), 2088(SA4), 2089(SA4), 2090(SA4), 2091(EM3), 2092(EM3), 2093(EM3), 2094(EM3), 2095(EM3), 2096(EM3), 2097(HY7)[c], 2098(SA4)[c], 2099(EM3)[c], 2100(AT3), 2101(AT3), 2102(AT3), 2103(AT3), 2104(AT3), 2105(AT3), 2106(AT3), 2107(AT3), 2108(AT3), 2109(AT3), 2110(AT3), 2111(AT3), 2112(AT3), 2113(AT3), 2114(AT3), 2115(AT3), 2116(AT3), 2117(AT3), 2118(AT3), 2119(AT3), 2120(AT3), 2121(AT3), 2122(AT3), 2123(AT3), 2124(AT3), 2125(AT3).

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NOVEMBER: 2241(HY11), 2242(HY11), 2243(HY11), 2244(HY11), 2245(HY11), 2246(SA6), 2247(SA6), 2248(SA6), 2249(SA6), 2250(SA6), 2251(SA6), 2252(SA6), 2253(SA6), 2254(SA6), 2255(SA6), 2256(ST9), 2257(ST9), 2258(ST9), 2259(ST9), 2260(HY11), 2261(ST9)[c], 2262(ST9), 2263(HY11), 2264(ST9), 2265(HY11), 2266(SA6), 2267(SA6), 2268(SA6), 2269(HY11)[c], 2270(ST9).

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FEBRUARY: 2355(CO1), 2356(CO1), 2357(CO1), 2358(CO1), 2359(CO1), 2360(CO1), 2361(PO1), 2362(HY2), 2363(ST2), 2364(HY2), 2365(SU1), 2366(HY2), 2367(SU1), 2368(SM1), 2369(HY2), 2370(SU1), 2371(HY2), 2372(PO1), 2373(SM1), 2374(HY2), 2375(PO1), 2376(HY2), 2377(CO1)[c], 2378(SU1), 2379(SU1), 2380(SU1), 2381(HY2)[c], 2382(ST2), 2383(SU1), 2384(ST2), 2385(SU1)[c], 2386(SU1), 2387(SU1), 2388(SU1), 2389(SM1), 2390(ST2)[c], 2391(SM1)[c], 2392(PO1)[c].

MARCH: 2393(IR1), 2394(IR1), 2395(IR1), 2396(IR1), 2397(IR1), 2398(IR1), 2399(IR1), 2400(IR1), 2401(IR1), 2402(IR1), 2403(IR1), 2404(IR1), 2405(IR1), 2406(IR1), 2407(SA2), 2408(SA2), 2409(HY3), 2410(ST3), 2411(SA2), 2412(HW1), 2413(WW1), 2414(WW1), 2415(HY3), 2416(HW1), 2417(HW3), 2418(HW1)[c], 2419(WW1)[c], 2420(WW1), 2421(WW1), 2422(WW1), 2423(WW1), 2424(SA2), 2425(SA2)[c], 2426(HY3)[c], 2427(ST3)[c].

APRIL: 2428(ST4), 2429(HY4), 2430(PO2), 2431(SM2), 2432(PO2), 2433(ST4), 2434(EM2), 2435(PO2), 2436(ST4), 2437(ST4), 2438(HY4), 2439(EM2), 2440(EM2), 2441(ST4), 2442(SM2), 2443(HY4), 2444(ST4), 2445(EM2), 2446(ST4), 2447(EM2), 2448(SM2), 2449(HY4), 2450(ST4), 2451(HY4), 2452(HY4), 2453(EM2), 2454(EM2), 2455(EM2)[c], 2456(HY4)[c], 2457(PO2)[c], 2458(ST4)[c], 2459(SM2)[c].

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JUNE: 2494(IR2), 2495(IR2), 2496(ST6), 2497(EM3), 2498(EM3), 2499(EM3), 2500(EM3), 2501(SM3), 2502(EM3), 2503(PO3), 2504(WW2), 2505(EM3), 2506(HY6), 2507(WW2), 2508(PO3), 2509(ST6), 2510(EM3), 2511(EM3), 2512(ST6), 2513(HW2), 2514(HY6), 2515(PO3), 2516(EM3), 2517(WW2), 2518(WW2), 2519(EM3), 2520(PO3), 2521(HY6), 2522(SM3), 2523(ST6), 2524(HY6), 2525(HY6), 2526(HY6), 2527(IR2), 2528(ST6), 2529(HW2), 2530(IR2), 2531(HY6), 2532(EM3)[c], 2533(HW2)[c], 2534(WW2), 2535(HY6)[c], 2536(IR2)[c], 2537(PO3)[c], 2538(SM3)[c], 2539(ST6)[c], 2540(WW2)[c].

c. Discussion of several papers, grouped by divisions.

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